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APPLICATIONS FOR ADMISSION AND TRANSFER

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

SOME SOIL PRESSURE TESTS

BY H. DE B. PARSONS[1], M. AM. SOC. C. E.

SYNOPSIS

This research is a sequel to a previous paper entitled "Hydrostatic Uplift in Pervious Soils".[2] Measurements were made of the horizontal pressures on a stationary bulkhead, resulting from bank sand and bank gravel. The fills were 7 ft deep, and alternately dry, fully saturated with water, and drained. The results indicated that the horizontal pressures for a pervious soil are more dependent on the angle of internal friction than on the angle of repose. At the beginning of irrigation, the pressures became less until the hydraulic pressure, due to the rising water, began to increase more than the relatively diminishing pressures from the soil.

GENERAL

This research was made to verify the horizontal pressure due to a fill completely saturated with water. Such conditions may occur behind retaining walls and dams. Writers of textbooks have assumed that the horizontal pressure equalled the hydraulic pressure plus the soil pressure when using the weight of soil in water and not in air. These tests on sand and fine gravel indicated that the value of ϕ should be that of the angle of internal friction and not the angle of repose. The tests also developed a novel and interesting phenomenon, namely, at the beginning of irrigation the horizontal pressures (due to the fill and to the water near the bottom) first decreased and then began to increase as the hydraulic pressure from the rising water increased. This change of pressure will be mentioned more fully later.

The testing apparatus used in this investigation is described in detail at the end of this paper. The front end of the testing apparatus was closed by a suspended bulkhead, sealed at the edges with painted canvas (see Fig. 1). This bulkhead was supported by steel rods and links, and was carefully balanced to hang vertically after the scales were attached. When in position, the lower edge was $\frac{1}{4}$ in. above the floor of the apparatus. The area exposed to the fill was 4.926 ft wide by 6.979 ft high. Three scales, *A*, *B*, and *C*, of

NOTE.—Discussion on this paper will be closed in February, 1934, *Proceedings*.

[1] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

[2] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1317.

the lever and movable-weight type, with knife-edges and ball-bearings, were attached to its outside face. Each scale could weigh 3 000 lb, or a total of 9 000 lb for the set. The scales were located so that each might take practically the same loading when the apparatus was full. The end levers had a freedom of movement at their tips of $\frac{1}{2}$ in, and the measuring leverage was 125 to 1. In consequence, movement of the bulkhead was restricted to 0.004 in. Therefore, the bulkhead can be considered as stationary at all loadings.

Fig. 1 illustrates the face of the bulkhead exposed to the pressure of the fill, and also gives a side view with the scales attached. The crosses on the face (Points *A*, *B*, and *C*, Fig. 1 (*b*)) mark the centers of the scale attach-

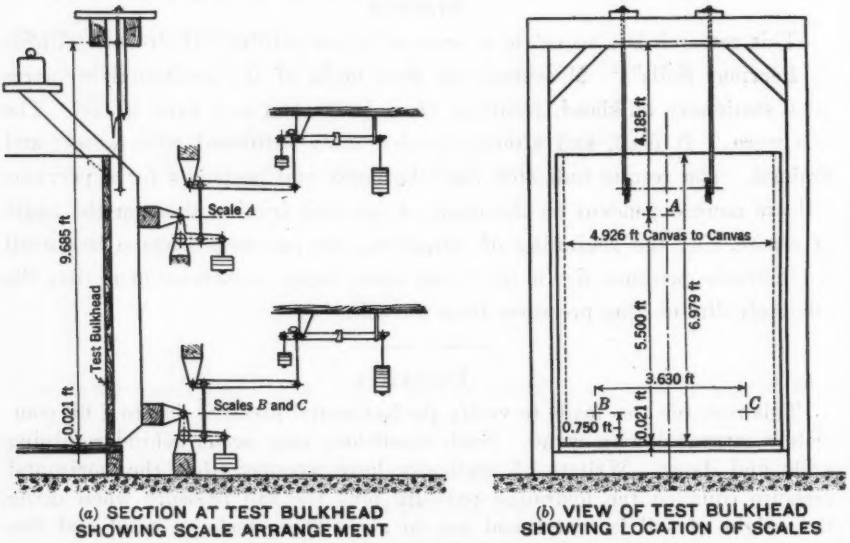

FIG. 1.—TESTING BULKHEAD, SHOWING SECTION AND FACE

ments. The dimensions are in feet, as determined from the finished bulkhead. The canvas, sealing the edges, is not shown, but will be described later. The sum of the three scale readings gave the horizontal pressure, and the center of the resultant pressure was determined from the individual scale readings. The calculations, by moments and levers, to find the centers of pressure were made always from the links, while the distances from the centers to the bottom of the bulkhead were found by subtraction.

The apparatus was standardized by filling it with water. A series of tests were made with water at various depths on the bulkhead, and the calculated results at each depth (Column (1)) are given in Column (3) and the averaged results by the scales in Column (5), of Table 1. The theoretical hydraulic pressures, using weight of water at 62.22 lb, on the area of bulkhead found by multiplying the actual depth by a width of 4.926 ft, were approximately 5.1% greater than the sums of the scale pressures, Column (5); that is, 5.1% of the water pressure was required to overcome the stiffness

of the bulkhead and the influence of the canvas sealing its edges. It was also found that the actual location of the center of pressure fell somewhat below the theoretical, indicating a reduction in the head acting on the bulkhead. To compensate for the reduced pressure and to harmonize the discrepancy in the location of the center of pressure, it was assumed that the water acted on an "effective" area 5.1% less than its total area exposed to the water, As both the width and height had to be diminished, this assumption results in a slight error which, however, was thought permissible.

By inspection, the effective areas had an approximate width of 4.75 ft, and effective depths of 0.073 ft less than the actual ones. These reductions in dimensions would not have remained constant for shallower depths of water, but that is immaterial as only the actual depth (6.979 ft), was used in the soil tests.

TABLE 1.—STANDARDIZING TESTS; WATER PRESSURES—CALCULATED AND BY SCALES

HEAD OF WATER ON BULKHEAD, IN FEET		Theoretical water pressure, in pounds	PRESSURE ON BULKHEAD, IN POUNDS		CENTER OF RESULTANT PRESSURE ABOVE BOTTOM OF BULKHEAD, IN FEET	
Actual (1)	Effective (2)	(3)	Reduced from theoretical (4)	Average by scales (5)	Calculated (6)	Average by scales (7)
3.979	3.906	2 390	2 268	2 272	1.302	1.321
4.979	4.906	3 784	3 590	3 575	1.635	1.656
5.979	5.906	5 469	5 190	5 155	1.968	1.978
6.979	6.906	7 460	7 079	7 080	2.302	2.300

In Table 1, the effective heads of water, given in Column (2), were determined by deducting 0.073 ft from the actual head in Column (1). The figures in Column (4) represent the water pressures on the effective areas, while those in Column (5) were obtained from the averaged scale readings. The figures in Column (6) are one-third of those in Column (2), while the figures in Column (7) are the averaged results of calculating the centers of resultant pressures from the scale readings. A comparison of the figures in Column (4) with those in Column (5) and, also, of the figures in Column (6) with those in Column (7), show the agreement between the working of the scales and the calculations based on the effective area of the bulkhead.

To get the true pressure of the fill in any test, the sum of the pressures as recorded by the scales should be increased in the ratio of 100 to 94.9. This "corrected pressure" should be divided by 4.926 ft, the width of the bulkhead from canvas to canvas, to get the "corrected pressure" per foot width, as mentioned later.

The horizontal pressures recorded are for dry, saturated, and drained fills, and all had a depth of 7 ft, which, with the $\frac{1}{4}$-in. clearance under the bulkhead, made the 6.979-ft depth on the bulkhead. While the scales recorded the pressures due to the dry and drained (wet) fills, those for the saturated fills were the sum of the full hydrostatic pressure plus the pressure due to the irrigated soil.[2] The recorded pressures do not include frost action, as freezing creates another set of conditions and likewise of pressure.

Many of the calculations were made with the slide-rule. When made numerically, long decimals were shortened. Soil tests, such as those described in this paper, will not repeat with strict accuracy, and only a close approximation was essential.

Materials Tested

The soils tested were bank sand and bank gravel. Three samples of each were passed through a set of Tyler sieves, and Fig. 2 shows the mechanical analyses in graphic form. It will be noted that the bank gravel contained a

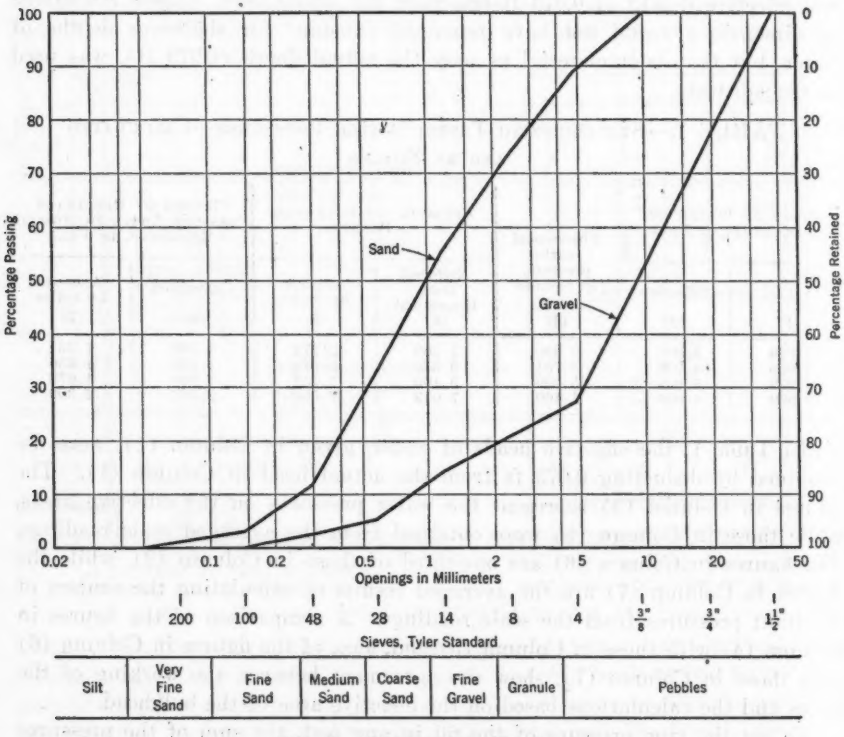

FIG. 2.—MECHANICAL ANALYSES

considerable proportion of small particles. Samples were taken of the bank sand as received, and its loss of weight (moisture) was determined by drying at 5.45 per cent. With the gravel, the loss was about 5.5 per cent. All the tests herein recorded as "dry" really contained these percentages of moisture.

Careful tests were made to find the weights and other characteristics of the sand and gravel, which are given in Table 2.

The samples were "rodded" according to the rules of the American Society for Testing Materials, Specifications C–29–27. The samples were rammed by a 20-lb bar, dropped 6 in. The bottom of the bar had an area of 6 sq in., in order to conform with the "rammed" material in the pressure

tests, in which the material was placed in 1-ft layers and compacted with a 50-lb tamp with a fall of 6 in., each layer being rammed twice. The tamp was always used with the flat of the base parallel to the slope; that is, the handle was at a right angle to the surface of the slope.

TABLE 2.—GENERAL DATA FOR MATERIALS TESTED

Description	Sand	Gravel
Weight of loose material, in pounds per cubic foot	101.5	115
Weight of rodded material, in pounds per cubic foot	111	122
Weight of rammed material, in pounds per cubic foot	112.5	122
Weight in water, in pounds per cubic foot	67.75	74.5
Specific gravity	2.59	2.60
Percentages of voids on rodded material	30.48	23.55
Weight of water, in pounds per cubic foot, at average of temperatures used	62.22	

The low percentage of voids in the gravel was due undoubtedly to the fact that the gravel was well graded, the spaces between the larger particles being filled with the finer material.

PRESSURE TESTS

Seven series of tests were made, with seven tests in each series, or forty-nine in all. In each series, the soil was put into the testing apparatus, without surcharge, its surface being always level and 7 ft above the bottom of the apparatus. It was then alternately irrigated and drained. The water was allowed to enter the apparatus in the open space behind the cross-partition; it rose at an approximate rate of 1 ft in 5 min, and this space was kept full until the water became flush with the surface of the soil. Records of the scale pressures were made before water was admitted, and afterward at intervals of from 3 to 5 min, until the soils were fully saturated and the scale readings became constant. The apparatus was then drained at the approximate rate of 6 in. in 5 min until 2 ft was drawn off, after which the rate was increased to 2 ft in 5 min, as measured in the open space. As before, the scale readings were taken at intervals of from 3 to 5 min, and continued until the readings became constant. This procedure was repeated for three irrigations and two drainings. At the end of the last irrigation, the apparatus remained full of soil and water for 18 hr, or longer, and then was allowed to drain for 12 hr. The water did not rise and fall at the bulkhead at the aforementioned rates, because it was admitted to, and drained from, the space behind the plank partition (see Fig. 4), and had to leak through the partition and seep through the soil (which required time) before it could rise and fall at the bulkhead. The time for a full irrigation was about 2 hr, and for a drainage about 1 hr and 15 min. There was also an interval of several hours between some of the drainages and the next irrigations, but these intervals had no appreciable effect on the results as the scale pressures were similar to those of tests between which there were no intervals.

In the first series, the sand was shoveled loose into the apparatus, without any effort to compact it. When the first irrigation was completed the sand

had settled 4 in. of its original 7-ft depth, or nearly 5 per cent. As this settlement altered comparisons with the results of the other six series, in which the soils were compacted, only a few results of loose, dry sand are reported, thus:

Depth of sand on bulkhead, in feet	6.979
Load on Scale *A*, in pounds	760
Loads on Scales *B* and *C*, in pounds	1 751
Total load on all scales, in pounds	2 511
Corrected total load, in pounds per foot of width	536
Center of pressure above bottom of bulkhead, in feet	2.187
Theoretical center at one-third of effective depth, in feet	2.302
$\tan^2 (45° - \frac{1}{2}\phi)$	0.218
Angle ϕ	40°

Shoveling the sand into the testing apparatus probably increased its weight per cubic foot in the lower portions and thus brought the center of pressure below the theoretical one-third of effective depth.

In each of the other six series of tests, the procedure was identical with the foregoing except that the apparatus was filled with the soil in 1-ft layers and each layer was compacted until the 7-ft depth was completed. After the last drainage, a final reading of the scales was taken at the end of 12 hr, and the series was ended.

TABLE 3.—PRESSURES, IN POUNDS PER FOOT OF WIDTH (CORRECTED)*

Series	Description of test (2)	Initial dry (3)	First irrigation (4)	First drainage (5)	Second irrigation (6)	Second drainage (7)	Third irrigation (8)	Final drainage (9)
2	Sand compacted in horizontal layers.	514	1 645 (135)	1 092	1 621 (111)	1 184	1 625 (115)	1 138
4	Sand compacted in horizontal layers.	524	1 678 (168)	1 057	1 657 (147)	1 075	1 640 (130)	1 166
3	Sand compacted in layers sloping from bulkhead	472	1 697 (187)	1 014	1 624 (114)	1 114	1 622 (112)	1 129
5	Sand compacted in layers sloping from bulkhead	536	1 625 (115)	1 054	1 624 (114)	1 133	1 606 (96)	1 162
6	Sand compacted in layers sloping toward bulkhead	572	1 656 (146)	964	1 626 (116)	1 044	1 630 (120)	1 077
7	Gravel compacted in horizontal layers	456	1 729 (219)	978	1 683 (173)	1 076	1 683 (173)	.....

* Values in parentheses are pressures due to the soil: that is, total pressures less the hydraulic pressure

These series were designated by numbers from 2 to 7, inclusive, and were compacted in the various ways described in Column (2), Table 3. In each series the compacting was done by the 50-lb tamp, as already described. The depth of the soil at the bulkhead was always 6.979 ft, and the effective head was 6.906 ft, as determined from the standardizing tests with water.

The measured pressures (Table 3) are those of the scales after the readings had become constant and showed that the repeated irrigations and drainages destroyed the effects from compacting. The 12-in. layers were too thick for any enduring effect under irrigations and drainages, and the results when the layers sloped either from or toward the bulkhead did not show any appreciably distinct characteristics from those when the layers were horizontal. The compacting, however, did consolidate the soils, as shrinkage, on application of water, was practically nil.

From an inspection of the results in Table 4, it will be seen that the horizontal pressure from the sand or gravel when dry (that is, as received from the bank and containing somewhat more than 5% of moisture), was 523 lb and 456 lb, respectively, per foot of width, but only 131 lb (1 641 − 1 510) and 196 lb (1 706 − 1 510) when fully irrigated; that is, saturated with water. These latter pressures are the differences between the saturated soil pressures and the water pressure only.

TABLE 4.—AVERAGE RESULTS OF TESTS

Item No.	Description (2)	Water only, no soil* (3)	Sand			Gravel		
			Dry (4)	Saturated (5)	Drained (6)	Dry (7)	Saturated (8)	Drained (9)
1	Total scale pressure on bulkhead, in pounds	7 080	2 453	7 691	5 052	2 136	7 998	4 816
2	Resultant pressure acting above bottom of bulkhead, in feet	2.300	2.236	2.296	2.273	2.343	2.356	2.532
3	Difference between point of resultant and one-third of effective height, in feet†	−0.002	−0.066	−0.006	−0.029	+0.041	+0.054	+0.230
4	Corrected‡ total pressure per foot of width, in pounds	1 510	523	1 641	1 077	456	1 706	1 027
5	Values of $\tan^2(45°-\frac{1}{2}\phi)$	.....	0.195	0.093	0.356	0.155	0.125	0.310
6	Angle of Internal Friction: Computed	.....	42° 10′	58° 0′	27° 10′	46° 40′	53° 10′	30° 0′
	By experiment	.....	48° 40′	61° 40′	.....	48° 0′	58° 0′	.....

* For comparison. † Positive values greater than, and negative values less than, the theoretical.
‡ An allowance of 5.1% was made for stiffness of bulkhead and influence of canvas at edges.

As the apparatus stood long enough for the water to saturate fully the mass (by rising from the bottom to the top), the water acted on the bulkhead both through the voids and by transmitting pressure through the soil particles. Therefore, the excess pressure recorded by the scales over that due to water only, was the pressure from the weight of the soil. Applying the Rankine or Coulomb theory, for a fill without surcharge, the angles between the horizontal and the planes of internal friction for the dry soils and for the fully saturated soils varied. The angles always were smallest for the drained soils, intermediate for the dry soils, and largest for the saturated soils. It will be remembered that the intensity of the horizontal pressure varies inversely with the angle of internal friction.

The figures in Item No. 5, Table 4, were determined from the equation, horizontal pressure $= \frac{1}{2}\, w\, h^2 \tan^2(45° - \frac{1}{2}\phi)$, in which the pressures are given in Item No. 4, the values of w in Table 2, and h taken as the effective heads. In the drained conditions, water remained in the interstices between the particles, although drained for 12 hr, and made the soils heavier, which

would increase the horizontal pressures. For the drained conditions, an allowance for increase in weight was made by assuming that one-half the voids were full of water; that is, for the drained sand the weight was taken at 120.4 lb per cu ft, and for the gravel at 129.3 lb. An error in this assumption will not cause a serious change in the results quoted. It was not necessary to be too exact, as soils are not homogeneous, but differ throughout their mass. The computed angles of internal friction, Item No. 6, Table 4, correspond to the tangents in Item No. 5. It is difficult to duplicate the exact conditions of a large test mass of a soil in a much smaller sample, but an effort to do so was made which gave the angles "by experiment," as mentioned in Item No. 6. This was done by putting the compacted soils in two boxes and measuring the force to pull one over the other, allowances being made for the weight of the upper box and for the friction between the boxes. This method gave the tangents corresponding to the angles quoted. The soils were tested both dry and saturated.

During drainage the velocity of the water between the particles must have been appreciable. It would seem that the retreating water removed some support from the particles, which it maintained while the mass was saturated in a quiescent state. On the next irrigation, the apparent support of the particles by the water was restored.

The recorded scale pressures developed an interesting phenomenon as irrigation of the soil progressed, which the writer believes to be novel, and reported for the first time. After an initial reading of soil pressure was made, and the admission of water commenced, the combined pressure decreased until the water rose about 1 ft, or somewhat less, in the soil. Then, as water continued to rise in the soil, the pressure increased and continued to increase until complete saturation had taken place, with water standing

TABLE 5.—SCALE PRESSURES IN TWO TESTS AS WATER RISES IN THE SOIL, IN POUNDS

SAND			GRAVEL		
Scale *A*	Scales *B* and *C*	Total	Scale *A*	Scales *B* and *C*	Total
821	1 860	2 681	716	1 420	2 136
.....	.....	.....	702	1 387	2 089
812	1 847	2 659	708	1 650	2 358
808	1 804	2 612	720	1 772	2 492
800	1 796	2 596	850	2 317	3 167
1 015	2 960	3 975	2 300	5 097	7 397
2 250	5 013	7 263	2 698	5 310	8 008
.....	.....	.....	2 730	5 375	8 105
2 590	5 172	7 762	.....	.....	.....

at the level of the top of the fill. This phenomenon was repeated in most of the irrigation tests, and Table 5 gives the scale records for two of the tests.

The height of soil on the bulkhead was constant at 6.979 ft, while the depth of water was variable, rising from 0 ft at the initial pressure record to the top of the soil at the last record. It is not clear why this fall of pressure at the beginning of irrigation should occur. As a possible explanation the following suggestion is offered: Below the water level, the soil weight would be lighter (due to its weight in water), while above the water level,

the soil would retain its dry weight. This would reduce the horizontal pressure due to the soil. As the irrigation-water level rose, the increase in hydrostatic pressure would be greater than this reduction of soil pressure. Furthermore, introduction of the water increased the internal angle of friction of the soil, resulting in a decrease of the volume of the soil wedge causing the horizontal pressure. This reduction in pressure from the soil would be relatively greater when the water levels were low and the hydrostatic pressure was small.

When drainage commenced, the pressure fell until about 2 ft of water remained in the soil, and, thereafter, it decreased very slowly until the final drained reading was recorded. A hypothetical calculation of the pressures, as drainage progressed, indicated that the regimen of decrease in pressure would be similar to that for rise in pressure due to irrigation; that is, the pressure would decrease below and then increase to the final drained pressure. The hypothetical calculation was based on the assumption that, at any intermediate stage of the water level, the horizontal pressure was the sum of the pressure due to the dry fill above the water level, of the pressure due to the fill below the water level, and of the hydrostatic pressure. It was realized that the value of $\tan^2 (45° - \frac{1}{2}\phi)$ did not remain constant as the water rose, and a compromise was made by plotting the values for dry and saturated conditions, with water depths as ordinates, and joining the points by a straight line. Then, values for each foot of depth of water could be determined. Probably the change in values of the tangents did not vary as this linear assumption, but the interpolations were approximate.

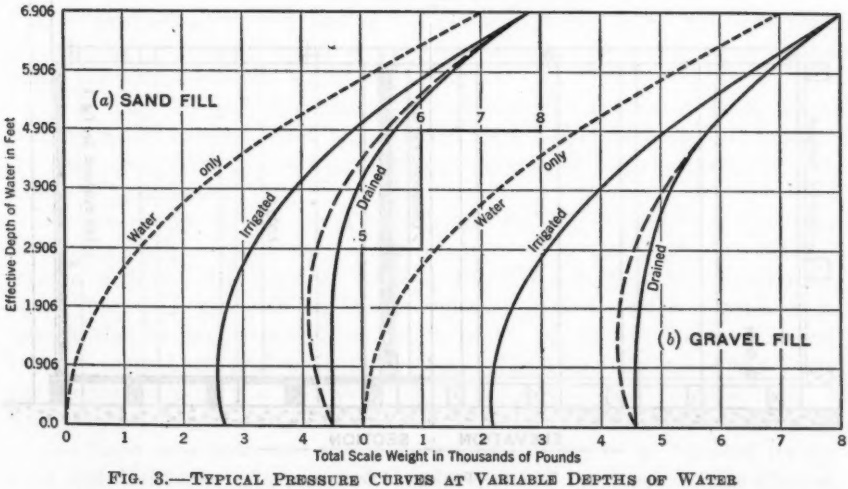

FIG. 3.—TYPICAL PRESSURE CURVES AT VARIABLE DEPTHS OF WATER

The pressures for Series 6 with sand and for Series 7 with gravel are illustrated in Fig. 3, on which curves of pressure due to water only have been drawn. The broken lines for the drainage curves are computed values. The horizontal distance between the water curve and either of the remaining curves, measures the pressure from the soil. It will be noted that the differ-

ence between the water curve and the irrigation curve steadily decreases as the water rises. In other words, the part of the total pressure due to the soil becomes less and less as irrigation progresses until, at full saturation, the pressure due to the soil is only a small percentage of that due to water alone. Furthermore, it is only a small portion of that due to the soil without any water (initial pressure).

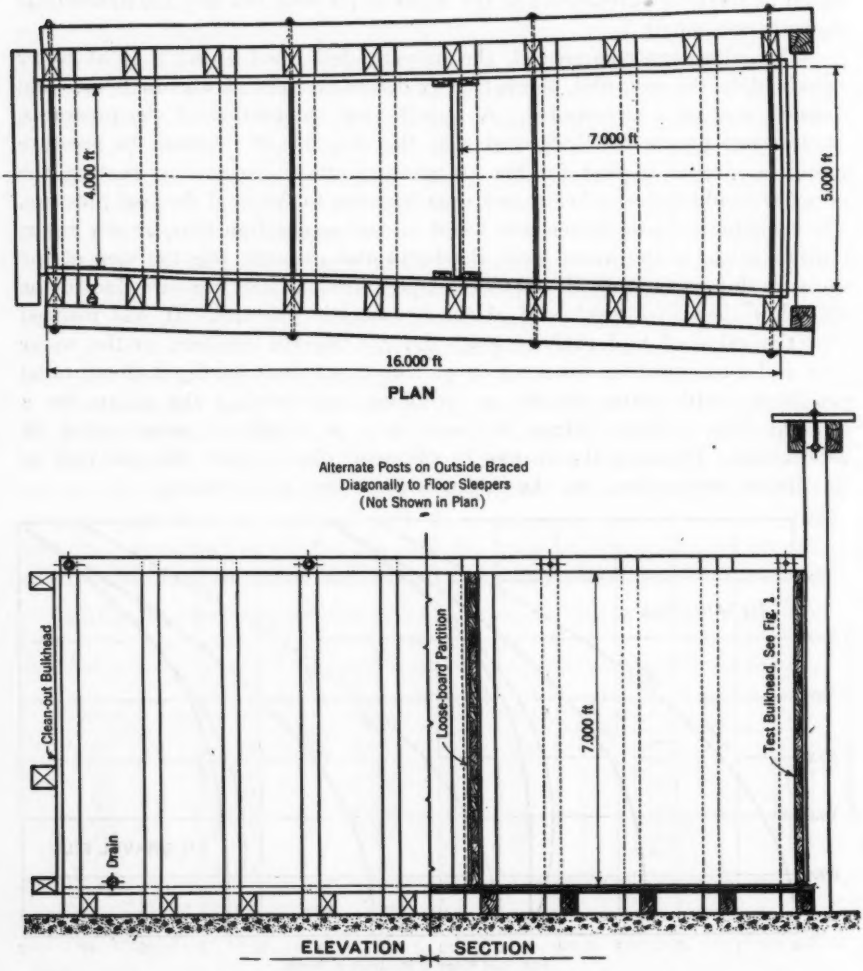

FIG. 4.—TESTING APPARATUS

DESCRIPTION OF APPARATUS

The testing apparatus, Fig. 4, was constructed of 2-in. dressed yellow pine planks, grooved and fitted with splines for water-tightness. It was stiffened by 6 by 4-in. uprights at the sides and sleepers across the bottom and by steel tie-rods across the top. The box stood on a level concrete floor, and

its interior width was tapered to reduce any arching of the soils. It was lined with building paper, and the front end was the free bulkhead shown in Fig. 1, suspended by two steel rods, linked into shorter rods having washers and nuts, that were supported by cross-pieces. The cross-pieces, in turn, were supported by braced posts. There was opportunity in the supporting arrangement to adjust the bulkhead to the front end of the testing box.

The supporting rods were practically in the plane of the center of gravity. The three scales were attached and supported, as shown in Fig. 1. To balance the bulkhead in a truly vertical position, counterweights attached to the top of the bulkhead were provided. The openings at the edges of the bulkhead were closed by canvas tacked to the sides and bottom, then miter-folded at the bottom corners, and tacked to the bulkhead. The canvas was in two layers and painted to prevent leakage. When the apparatus was empty and ready for testing, the scales were balanced at zero readings, and were very sensitive to pressure on the bulkhead. The rear end of the box was arranged to open for the easier removal of the soils between each series of tests. The box was designed long enough for a more extended research, but such additional work was not undertaken. To reduce the quantity of soil handled, a loose plank partition was placed 7 ft back from the bulkhead as shown in Fig. 4. This was far enough so as not to interfere with the planes of rupture of the soils. Water for irrigation was admitted to the empty space in the rear of this partition, and it flowed through the open joints and the soil to the bulkhead. There was also a valve for drainage.

Summary

1.—There are many variables in soil problems, and only by the agreement of a number of researches can actual facts become established, which is the excuse for publishing this work.

2.—It may be said that "arching" affected the results of these tests. Arching might have occurred in the "dry" tests, but the repeated irrigations and drainages which followed gave such uniform results that arching did not appear to be a factor.

3.—The results of these tests approximate the Rankine and the Coulomb theory for soils without surcharge, provided ϕ is the angle of internal friction and not the angle of repose. Field observations of unsupported banks, in which the soil had some moisture content, show a surface of rupture that is not a true plane, but is nearly vertical at the surface and becomes more horizontal near the bottom. Such observations confirm the test results, namely, that the wedge causing the pressure is smaller than in theory, which uses the angle of repose.

4.—These tests indicated that the center of pressure, on repeated irrigations and drainages, approximated the theoretical $\frac{1}{3}\,h$, except for the drained gravel where the center was considerably higher than the theoretical (see Table 4). It is not apparent why the point of resultant pressure was high for the gravel, especially in the drained state. It is conceivable that the plane of rupture in this coarser material was a warped or curved surface, which reduced the section of the wedge from the theoretical triangular shape

and placed its center of gravity above that of the triangle. The wet condition after drainage may have increased the cohesion of the gravel particles and thus produced a smaller wedge by a curved rupture plane.

5.—When the soil was fully irrigated, the pressure consisted of the hydrostatic pressure plus a pressure due to the soil. The portion of the pressure which was due to the soil was less than that which would have been expected when using the angle of internal friction as deduced from the dry material (that is, containing about 5% moisture), and making an allowance for the reduced weight of the material; that is, the ratio of its weight in water to its weight in air. It may be that under saturated conditions, the plane of rupture is curved, rather than flat, as explained herein under Summary Item 4.

6.—Under fully irrigated conditions it would seem from Table 4 that the pressure due to the sand was 131 lb (total pressure, 1 641 lb, less hydraulic pressure, 1 510 lb). This soil pressure was about 25% of the dry soil pressure, namely, 523 lb. In the same way, for the gravel, the total pressure was 1 706 lb less the hydraulic pressure, 1 510 lb, leaving 196 lb for the gravel portion of the total. This latter value is about 43% of the dry gravel pressure of 456 lb. It will be noted that these soil-pressure percentages are much less than the percentages of the weight of the sand or gravel in water to their weight in air, which were about 61 in both cases.

7.—Under drained conditions, the horizontal pressures of the soils were high and the computed angles of internal friction were low, when compared to the results for dry and saturated conditions. This was probably due to the increased weight of the water retained in the interstices, and also to a possible hydrostatic pressure resulting from the "head" corresponding to the water thus held by the particles. Ordinarily, wet soils are stiffer than dry soils due to cohesion between the particles, but these drained soils possibly retained more water than necessary for maximum cohesive effect.

8.—The scale pressures as given in the horizontal lines of Table 5, were taken at similar intervals, and, therefore, the sequence of decrease and subsequent increase of pressures is comparable for both the sand and gravel tests. Probably the gravel, being the coarser, allowed the water to seep through its mass faster than through the sand and rise more quickly on the bulkhead. Thus, the lowest pressure for the gravel occurred at the first interval, after the initial, while for the sand the decrease continued until the fourth interval.

Acknowledgments

The writer acknowledges his thanks to the Trustees of Rensselaer Polytechnic Institute, Troy, N. Y., for partial financial assistance, and to Palmer C. Ricketts, Hon. M. Am. Soc. C. E., for space for the apparatus. He is indebted, furthermore, to T. R. Lawson, M. Am. Soc. C. E., and Professor Edward J. Kilcawley, for determining the weights of the soils; to Lt. Hunt V. Martin, C. E., U. S. N., a Post-Graduate Student at the Institute, for the manipulation of the tests; and to Eugene E. Halmos, M. Am. Soc. C. E., for assistance in the analytical work.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

LINCOLN HIGHWAY FROM JERSEY CITY TO ELIZABETH, NEW JERSEY

BY SIGVALD JOHANNESSON,[1] M. AM. SOC. C. E.

SYNOPSIS

In order to take care of the vehicular traffic concentrating at the west end of the Holland Tunnel, between Jersey City, N. J., and New York, N. Y., the New Jersey State Highway Commission began in 1924 the construction of a "super-highway," leading from the tunnel through Jersey City, Kearny, Newark, and Elizabeth, to connection with existing or new State highways. The construction of this highway was completed in 1932.

As this "super-highway" necessarily had to go through built-up sections of cities, and through other sections highly developed for railroad and industrial purposes, and as it was expected to carry a large volume of traffic, it was decided at the inception of the work to build it so as to avoid grade crossings throughout the greater part of its length.

In this paper are discussed the general aspects of the design of this highway, which included embankments, viaducts, high-level bridges, and a subway, with their ramp connections to the surface streets.

GENERAL DESCRIPTION

The last link of the New Jersey State Highway, designated the Lincoln Highway, leading from the west end of the Holland Tunnel through Jersey City, Kearny, Newark, and Elizabeth, to connection with the main highways to Trenton, Philadelphia, Pa., and beyond, was completed eight years after the inception of the work, and was opened for traffic on November 24, 1932. This highway, the alignment and partial profile of which are shown in Figs. 1 and 2, respectively, is about 12 miles long, and its cost has been approximately $40 000 000.

At present (1933) this highway carries traffic at the rate of 11 000 000 vehicles per year and is expected to become one of the most heavily traveled

NOTE.—Discussion on this paper will be closed in February, 1934, *Proceedings*.

[1] Designing Engr., State Highway Comm., Trenton, N. J.

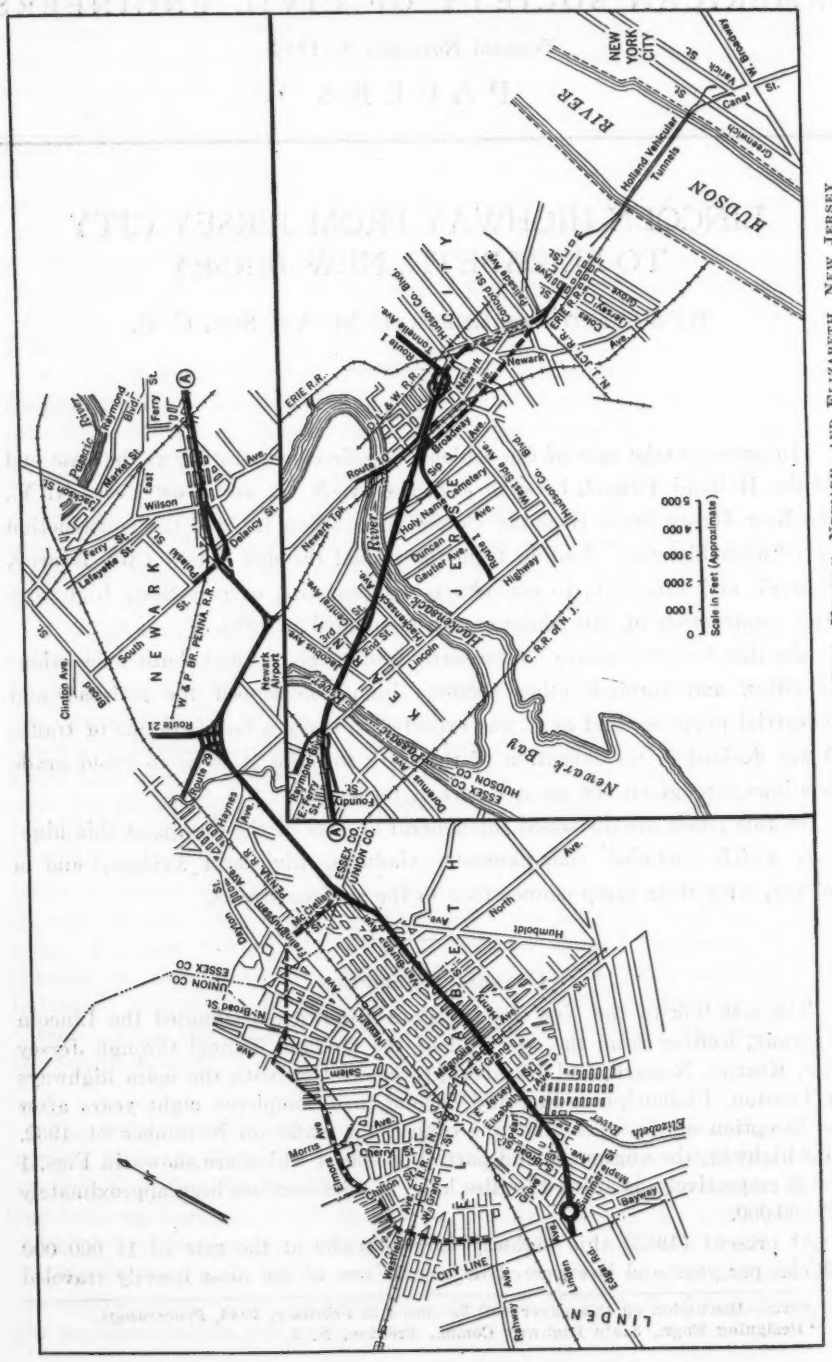

FIG. 1.—ALIGNMENT OF ROUTE 25 THROUGH JERSEY CITY, KEARNY, NEWARK, AND ELIZABETH, NEW JERSEY

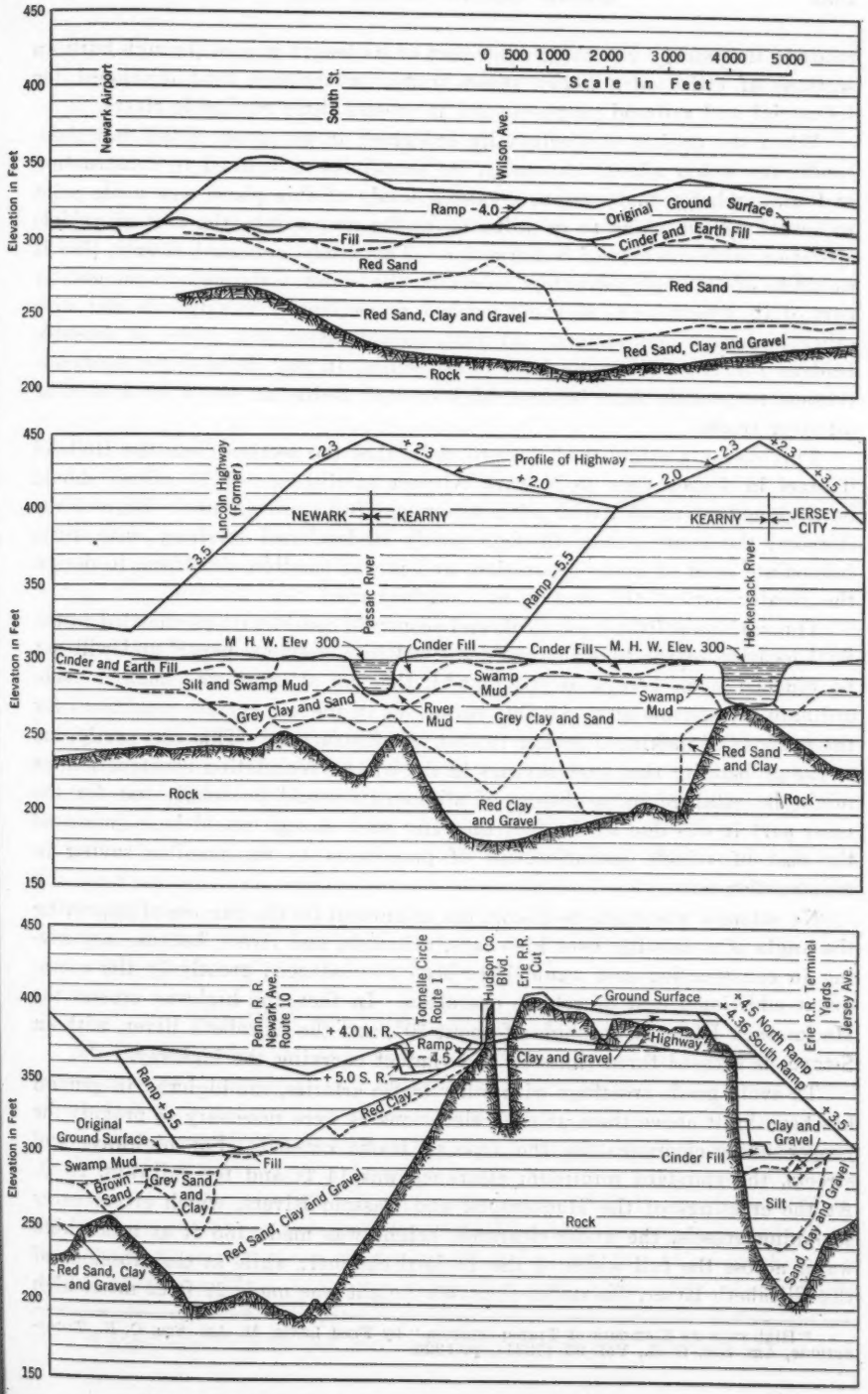

FIG. 2.—PROFILE FROM HOLLAND TUNNEL TO NEWARK AIRPORT

FIG. 1.—ALIGNMENT OF ROUTE 25

roads in the world. For the greater part of its length it goes through built-up sections of cities with heavy street traffic, or through land developed for industrial and railroad purposes, and it crosses three navigable rivers.

When the project was originally conceived it was appreciated that these conditions would affect, materially, its economics as applied to construction and use, and, for that reason, a careful study of this phase was made prior to actual design, so as to co-ordinate, as far as possible, the cost of vehicle operation with the cost of construction. This study indicated clearly, that it would be of economic advantage to design the highway throughout the greater part of its length so as to avoid grade intersections with railroads and with other highways and streets. At first, consideration was given to movable bridges for river crossings but, subsequently, it was decided for economic reasons to provide fixed bridges, high enough above the water level to clear all river traffic.

This decision established the principle, that the section from the Holland Tunnel in Jersey City to Newark Airport (a distance of 7¼ miles) should be designed so as to avoid all interference from cross-traffic. Beyond the Airport, the route passes through partly undeveloped territory with little immediate need of grade separation, and it was possible, therefore, to design the greater part of this section as a surface road.

The various political, physical, and economic considerations that led to the final location of the highway are described in a previous paper[2] and will not be touched upon herein except to add, that no attempt was made to subordinate the alignment to possible variations in the sub-surface conditions for the purpose of making a saving in cost of construction. This was partly due to the probability that such savings in the cost of foundation construction, as might be attained by a change of alignment, would be small; but for the most part it was due to the fact that any such change would have increased the cost of vehicle operation out of proportion to the possible saving in construction cost.

No attempt was made to distort the alignment for the purpose of improving the angle of a crossing over highways, railroads, and rivers, because any saving in construction cost would have been overbalanced greatly by the consequent additional cost of vehicle operation. In fact, the highway crosses the Hackensack River at an angle of about 50°, and the Elizabeth River, with an S-turn, is crossed three times by the viaduct carrying the highway.

To avoid grade crossings with other traffic arteries, the highway in general has been built above them at such elevations as were necessary to provide the proper under-clearance for the type of traffic crossed. For highways and streets, the standard minimum clearance was 14 ft and for railways, 23 ft. At the crossings of the Hackensack and Passaic Rivers, which rivers carry sea-going vessels, the under-clearance height was made 135 ft at mean high water across the full width of the dredged channel; while at the crossings of the Elizabeth River, the under-clearance height was made 22 ft at mean high

[2] "Highways as Elements of Transportation," by Fred Lavis, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1020.

water. At the crossing points, the Elizabeth River is only a small stream with practically no channel depth at low water, but, nevertheless, it is classified as a navigable river.

An exception to the general method of locating the highway above the surface traffic was made in Jersey City, where it crosses a hill about 100 ft high and about $\frac{3}{4}$ mile wide. In order to avoid excessive rise and fall on the profile with the consequent additional cost of vehicle operation, the line at this place was driven through a subway below the surface streets.

As previously stated, there are no grade intersections with any kind of traffic routes between the Holland Tunnel and Newark Airport; but, with the exception of certain isolated places, beyond the Airport there are no railway crossings, and the intersecting vehicular traffic is not of sufficient importance to warrant grade separation. The greater part of the line between Newark Airport and the southern boundary of the City of Elizabeth, therefore, is designed and built as a surface road.

In due time, however, it is probable that the vehicular traffic will develop sufficiently to justify a grade separation. For this reason a strip of land, 120 ft wide, has been acquired throughout this section. At present, the roadway occupies the central 50-ft. strip of this right of way. The remainder, which is intended for sidewalks and local roadways, is separated from the highway by narrow longitudinal "islands." When the crossing traffic has developed sufficiently to warrant the change, it is contemplated to raise the grade above the cross-traffic level, leaving the local roadways at the present grade to serve the surface traffic along the line of the highway.

Physical, topographical, and economic conditions have necessitated the use of many types of structures. Generally none of these types is novel or unusual in itself, but many of the designs, which now seem entirely obvious and simple, were the final results of many studies with various plans. The work on the highway was started in 1924 and completed in 1932, and during the intervening years, design was continuously in progress on some section of the highway. In details, the design of similar structures was not always identical if they were developed at different periods, because of new ideas and additional knowledge and experience gained.

Alignment and Profile of Roadway and Ramps

At the beginning of the work the width of the roadway was established at approximately 50 ft. In the earlier designs the width actually was made 48.5 ft, but, later, the roadway was made 50 ft wide. The maximum gradient was established at 3.5%, which value was maintained throughout the work for the main roadway. At grade breaks vertical curves were introduced, which were made quite long in the earlier period; but it was found difficult at times to provide proper drainage with these long curves, and gradually it became customary to make the length 67 ft for each 1% of grade change at ridges, and from 33 to 50 ft at valleys. In special cases, where these lengths have been decreased, because of physical conditions, an easy riding surface and a sight distance of not less than 500 ft have been maintained.

It was desired from the beginning, that the radii of horizontal curves should not be less than 1 000 ft., and, generally, this minimum requirement was maintained, although in some cases it was made less. For the designs made during the more recent years, the tendency was to increase the minimum radius to 1 500 ft, and, whenever possible, the curve radii have been made from 5 000 to 10 000 ft.

Ramp connections have been provided at intervals, leading from the adjacent roadway surfaces to the new level; and usually they enter at the middle of the highway, which is widened and divided where such ramps occur. At first, the reverse curves at these wide sections were generally made with radii of 1 000 ft. In one case, where local conditions were unfavorable, the reverse curves were designed with radii of less than 500 ft, but the result was not satisfactory. Later, the radii were increased to 2 000 ft for the convex part, and 1 000 ft for the concave part, of the reverse curve, which permitted the traffic to move with greater smoothness and safety, and the additional cost of construction was generally relatively small.

Wherever it was possible, it was found advantageous to place the head of a center ramp at a curve in the main line in order to avoid reverse curves. Such an arrangement is indicated in Fig. 3, which shows the outline of the

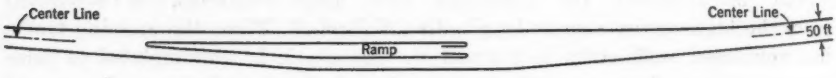

FIG. 3.—OUTLINE OF RAMP CONNECTION, KEARNY, N. J.

ramp connection in Kearny. The curve angle of the alignment at this point is small, but it was possible, nevertheless, to construct the roadways without reverse curves.

It was necessary, of course, to introduce a vertical curve in the profile at the head of a ramp. Where the profile of the main line is descending in the same direction as the ramp, there is no difficulty, but if it is pitched in

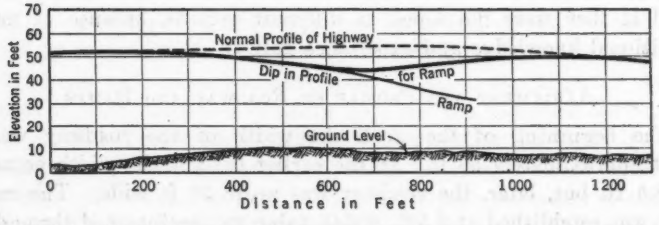

FIG. 4.—DIP IN NORMAL PROFILE TO PROVIDE FOR RAMP CONNECTION

the opposite direction, it is necessary to continue the rise on the ramp for some distance. This may add materially to the length, and, consequently, to the cost of the ramp. For the sake of economy, it was found advisable in certain cases, therefore, to dip the profile of the main highway at the head of the ramp. Such a dip was made near Newark Airport, as shown in Fig. 4, and a material saving in construction cost was gained by this expedient; but

the appearance of the viaduct, as seen from the side, is not as satisfactory as might be desired, because the reason for the dip is not apparent.

Center ramps usually have been made wide enough for two lanes, one for entering, and one for leaving, traffic. The actual designed width of the ramps has been changed from time to time, but in the later designs it was 24 ft. In some cases the gradients have been made a maximum of 5.5%, in order to shorten the length of the ramp.

As it was intended that no U-turns should be made at the head of the ramps, each center ramp would serve incoming traffic in one direction, and outgoing traffic in the opposite direction only. The original plan of connections, therefore, contemplated that alternate ramps should serve the traffic in alternate directions, but in certain cases others were subsequently introduced at important street crossings.

This was done, for example, in Jersey City, where State Highway Route No. 1 leads from the George Washington Bridge south to Bayonne. When the highway was designed originally, the intention was to provide a ramp connection at this point for eastbound entrance and westbound exit. It was intended also to construct a surface highway leading west from the bottom of the ramp, so that, in fact, there would be a surface grade crossing at this point. As it was expected that the traffic would be heavy, a traffic circle was designed for the surface intersection to minimize as much as possible the inevitable delays as well as the chances of accidents. The diameter of the center line of the circular roadway was made 240 ft, which is rather small, but local conditions did not permit a larger diameter without heavy addi-

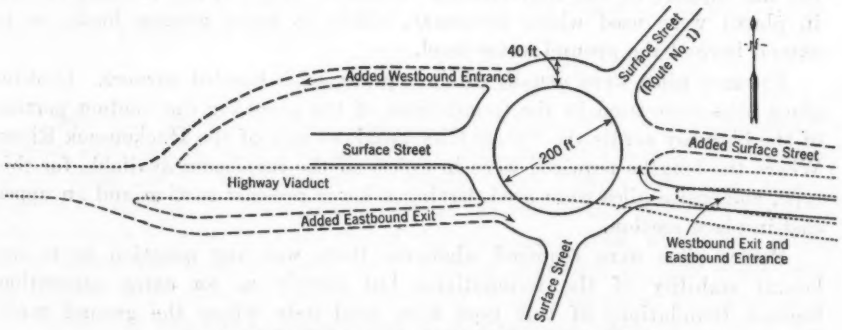

FIG. 5.—TRAFFIC CIRCLE, JERSEY CITY, NEW JERSEY

tional cost, and with a simple crossing this diameter should have been sufficient for satisfactory vehicle operation. A diagram of the traffic circle, with the originally contemplated street connections indicated in full lines, is shown in Fig. 5.

In the meantime, however, an additional street connection was deemed desirable on the east side (see dotted lines, Fig. 5), the introduction of which decreased the weaving distance necessary for proper vehicle operation, and, consequently, the efficient use of the circle. Later, it was decided further to add two side ramps from the west side of the traffic circle to the

highway viaduct so as to provide a direct westbound connection with Route No. 1, in addition to the eastbound connection (see dotted lines, Fig. 5).

It is quite apparent that with these additional roadways the traffic circle cannot function as originally intended; in fact, traffic control by signal lights and traffic officers has become necessary. With this control the flow of traffic is reasonably satisfactory, but it is probable that the circle would have been made larger or that some other arrangement would have been made, if all the roadway connections had been considered at the time of the original design.

There has been some doubt as to the advisability of using center ramps rather than side ramps, mostly on account of the delay that may occur when a slow-moving truck arrives at the head of the ramp and attempts to weave through the fast-moving inner-lane traffic to the lane alongside the curb. No doubt, some delay may occur at times; but, on the other hand, the traffic probably moves more smoothly if the curb line is not interrupted, as would be necessary if side ramps were used. In addition, the cost of the work is often materially increased with side ramps, particularly where the highway profile is high above the surface level.

Pile Foundations

As indicated in Fig. 2, the sub-surface conditions varied from place to place throughout the line of the highway. When rock or other firm materials was found near the surface level, the foundation problem usually was simple. Otherwise, it was frequently necessary to use either timber or concrete piles for the support of the foundations. Concrete piles (either pre-cast or cast in place) were used where necessary, either to carry greater loads, or to extend beyond the ground-water level.

Pre-cast piles were usually 16 in. square, with beveled corners. Cast-in-place piles were used in the foundations of the piers for the viaduct portion of the highway across the Jersey City meadows east of the Hackensack River. Where the lengths required were in excess of the maximum available for this type, composite piles were used, having a lower pre-cast section and an upper cast-in-place section.

Batter piles were required whenever there was any question as to the lateral stability of the foundations, but merely as an extra precaution because foundations of this type were used only where the ground could be expected to resist such lateral pressures as might be developed.

In one case, special arrangements were provided in the design to prevent displacement of foundations on concrete piles due to lateral pressures that might be caused by mud waves in the surrounding ground. This was in the meadow section in Jersey City, east of the Hackensack River, where the ground surface is just about the level of mean high water. The upper 10 ft or more of the meadow is soft silt, covered with vegetable growths, the roots of which form a tough fibrous mat. Below the silt are various mixtures of clay and sand, extending to rock, which is about 60 or 70 feet below the surface level.

The concrete foundations for this section were designed to extend 10 ft or more below the surface level and to be supported on concrete piles. With this arrangement, the top of the piles would generally be below the silt and the underlying material was expected to provide proper lateral support for the full length of the piles. It was thought, however, that future disturbance of the overlying silt, caused by filling in the meadows, or by other construction work in the vicinity of the highway, might possibly cause lateral mud waves, which would tend to displace the foundations. For this reason it was decided to include in the contract for the foundations a design for the two earth dikes, one at each side of the right of way of the highway, but outside the foundation sites. The design provided that strips of the vegetable mat, 18 in. wide, should be cut and removed at each side of each dike prior to the placing of the fill. This was done to prevent the fill from being suspended on the tough vegetable mat and to insure its settling through the silt, displacing and consolidating it so as to form dams, which would prevent mud waves from reaching the foundations.

As the dikes were used as roadways to provide access to the working site while the work was in progress, their construction is not believed to have added to the cost. If they had not been included in the contract, the Contractors would have had to make similar fills in order to provide access to their work, and the cost would necessarily have been included in some of the pay items under the contract. On the other hand, by including the construction of the dikes in the contract as a pay item, this necessary work was under the control of the Engineer, and the fill was placed in such a manner that it served a useful purpose, not only during the construction period, but also in the permanent structure.

Caisson Foundations

Where soft ground without material lateral stability was encountered, caisson foundations were used instead of piles, as, for example, at the foot of the steep east slope of the hill in Jersey City. Except for some surface fill, the borings at this place indicated that the sub-surface material was soft silt to within 8 or 10 ft of the top of the underlying rock, which was about 70 ft below the surface level. The viaduct structure at this point was to be about 80 ft above the surface level, and rather long spans were necessary on account of the track arrangement of the railroad yard, over which the viaduct was to be built. Taking all these matters into consideration, caisson foundations were deemed advisable.

The foundations were designed as cylindrical concrete piers, some of which were 8 ft and some 10.5 ft in diameter, and individual piers were used for each viaduct column to be supported. The caissons were designed as hollow concrete cylinders with an outside diameter equal to that of the finished pier and having a minimum wall thickness of 2 ft and a minimum specified reinforcement. After the caissons had come to a proper bearing, the construction was completed by filling them with concrete so that the finished structure was in effect a solid concrete cylinder.

A more extensive use of caisson foundations was made in the construction of the highway viaduct between and adjacent to the Hackensack and Passaic Rivers. The sub-surface strata, as indicated by borings, are shown in Fig. 2. It will be seen that the ground surface throughout this section is at or near the level of the water of the rivers and that the materials overlying the rock are soft and unsuitable for foundations. The surface of the rock, which at the east side of the Hackensack River is about 80 ft below the ground surface, rises rapidly toward the river. The original rock line at certain points under the river was perhaps less than 20 ft below the water level before the channel was deepened. West of the Hackensack River there is a rapid drop of the rock level which, near the west shore, is more than 90 ft below the water level. This somewhat peculiar formation appears to be due to a subterranean rock ridge, running in a north and west direction and intersecting the river at the location of the highway. The rock surface continues to drop until, at one point about midway between the Hackensack and Passaic Rivers, the level is about 140 ft below the ground (see Fig. 2). From this point the rock begins to rise again, so that at the crossing of the Passaic River the depth is about 60 ft, and beyond this river the elevation is from 50 to 60 ft below the surface.

Owing to the sub-surface conditions it was deemed necessary to use caisson foundations throughout the section from the east side of the Hackensack River to the west side of the Passaic River, and the design for the foundations was prepared accordingly. The drawings, however, did not show the details of the design of the caissons, but only the outlines of the finished work required, namely, concrete piers of certain horizontal cross-sections and extending from a given elevation near the ground surface to such a depth as would be necessary to attain proper bearing on sound rock. The reason for not showing the details was that, after all, it is the finished product that is desired, and as long as the work is finished in a safe and suitable manner, the method is generally of minor importance. In this way the individual contractor was also given an opportunity to design and construct the caissons according to such methods as his experience dictated might be most suitable and economical. The specifications provided, however, that,

"Detail drawings and descriptions of the caissons and of the methods and details of construction, which the contractor proposes to use, shall be submitted to the engineer for his approval before the proposed work is commenced."

The caisson work for this section was divided into two contracts and the two contractors submitted different designs for the caisson construction according to their individual preferences. One of them proposed a complete wooden caisson, with the idea of placing the caisson concrete during the sinking operation, and leaving the wooden sheathing permanently in place. The other preferred to construct section after section of a concrete caisson, removing the wooden forms before sinking. Both designs were acceptable to the Engineer, and in both cases the work was done satisfactorily.

For this work, as well as for all other caisson work, the specifications stipulated that compressed air should be used when the cutting-edge of

the caissons had reached a level a few feet above the rock surface, and that the caissons should be sealed to rock under compressed air. When the design was being prepared, consideration was given to the fact that, for some of the deeper caissons, the air pressure needed to seal the caissons might be greater than that permitted by law, if the ground should be found to be of an open character. An examination of the boring samples had indicated, however, that the material to be penetrated would be of a stiff, clayey, and impermeable character when exposed to the air pressure. It was believed, therefore, that if the work were done properly, the necessary air pressure would be materially less than the theoretical hydrostatic head indicated by the depth and that it would be within the legal limit. This belief was found to be correct, because the maximum air pressure used at any time during the construction of the caissons was 48 lb per sq in., and in the greater number of cases it was considerably less. The method of construction was to sink the caissons by open dredging until the cutting-edge was near the rock surface. The wells were then roofed over, and the remainder of the excavation was done under compressed air.

Owing to the depth to which the caissons were to be sunk, as well as to the height of the superstructure and to the character of the material to be penetrated, all the caisson foundations between the east side of the Hackensack River and the west side of the Passaic River were designed so as to provide a single pier for each viaduct bent. For the greater number of the piers the over-all horizontal dimensions were 25 ft wide by 60 ft long, while the piers for the main spans across the two rivers were 30 ft wide by 87 ft long.

On the west side of the Passaic River the conditions were somewhat different, as the average depth to rock was only about 60 ft, and the material overlying the rock was of such a character that it might be expected to afford some lateral stability. For these reasons each bent foundation was designed as two separate caissons, connected by means of a heavy concrete tie just below the surface level. The average size of these caissons was 14 ft wide by 16 ft long.

As the sub-surface conditions, as disclosed by the borings, indicated that either piling or caisson foundations might be used for this section, alternative designs were prepared and advertised for bids, one design involving caisson construction and the other concrete piling. The low bids received for the two plans were within 2% of the same amount, the bid for the caisson construction being the lower.

Subway Structure

As previously stated, the part of the highway that crosses the hill in Jersey City is built as a subway under the level of the existing surface streets (see Fig. 6). It was located north of, and quite close to, both the old Erie Railroad Tunnel, and the more recent cut made by the Erie Railroad Company for its passenger trains (see Figs 1 and 2). In fact, this location was selected largely in deference to the public opinion of the City of Jersey City,

which considered the Erie Cut as a scar across the face of the city. It was desired, if another cut were to be made across the hill, that this be adjacent and parallel to that already made, and the location was determined accordingly.

Borings made previous to the design of this subway indicated that the ground immediately underlying the surface consisted of clay and sandy clay;

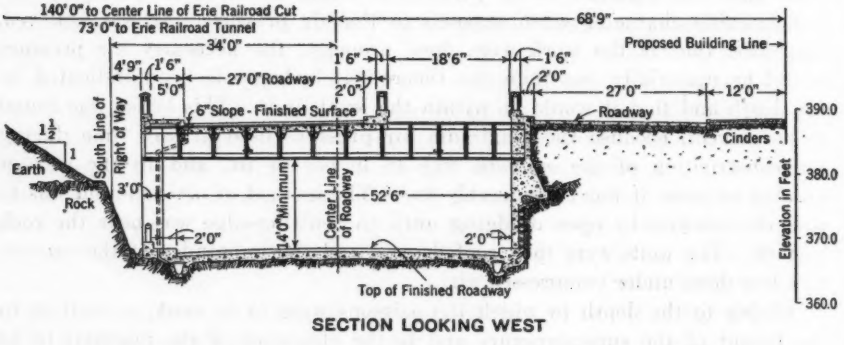

FIG. 6.—CROSS-SECTION OF SUBWAY, JERSEY CITY, NEW JERSEY

below this there was rock. As an average, a little more than one-half the depth of the subway excavation was indicated to be in soft ground, and the remainder in rock. The design provided for a concrete wall at the north side of the structure, the upper part of which was designed as a retaining wall for the soft ground, and the lower part as a facing wall for rock.

In order to provide proper ventilation, the excavation was sloped away from the highway at the south, but as it was desired to carry a surface road over the subway, concrete columns were provided at intervals on this side to support the roof of the subway. To provide additional ventilation facilities the roof was designed so as to cover only a part of the full width of the highway. All the steel work of the subway roof was designed to be covered with concrete, and reinforced concrete balustrades were provided around the ventilation opening at the north side of the structure, as well as along the south edge of the upper roadway.

In addition to the surface roadway constructed above the highway, another surface roadway was built north of, and immediately adjacent to, the subway. This roadway is used for westbound traffic while that built above the subway is for eastbound traffic, and together the two roadways form a wide, well paved, and much needed boulevard connecting the east and west parts of the city.

EMBANKMENTS

Earth embankments were used for the highway structure either where the roadway elevation is slightly above the surface level (as, for example, west of Newark Airport, where the elevation is only a few feet above the level of the meadow crossed), or where the physical conditions warranted embankments

rather than viaduct structures. A condition like this occurred in Newark, where for a distance of about 1½ miles the highway is parallel, and immediately adjacent to, an existing railroad embankment.

In order to avoid settlement the sequence of the construction work was usually arranged in such a manner that any embankment fill would be completed a year in advance of the pavement placed on it. Nevertheless, some settlement almost invariably occurs after the pavement is built, but usually this is not of any serious importance, except immediately adjacent to an abutment where an abrupt change of surface elevation will ocur. To compensate for this change of elevation, a reinforced concrete slab was introduced at the abutment during the earlier part of the work, which was capable of bridging a distance of about 20 ft. The design of this slab provided that one end of it would be supported on the abutment and the other on the earth embankment, the theory being that the end on the earth embankment might settle, but the concrete slab would form a runway to the higher elevation of the abutment, which would cause no inconvenience to traffic.

This arrangement in some cases did not prove satisfactory, although it would function as contemplated. The difficulty was that the settlement would cause changes in the roadway profile, which were too great to maintain comfortable riding conditions. The fact is that fairly small settlements are sufficient to produce excessive distortion of the profile in the short length available.

In later designs no attempt was made to introduce this concrete slab, but instead, the permanent pavement was omitted for a distance of about 50 ft immediately behind the abutment, and a temporary bituminous surface was used in its place.

Viaducts

Although concrete viaducts were used in a few cases, where the span lengths were moderate, the greater part of the viaduct structures was built of steel, preferably steel plate girders. The largest girder was 144 ft long, 11 ft 4½ in. deep, and weighed 60 tons.

Expansion was provided for mostly by means of double columns at the expansion joints; in fact, no sliding joints were used, throughout the length of the highway, where it was possible to avoid them. There were two reasons for attempting to avoid sliding joints, the first being that they are subject to wear and consequent expensive maintenance. Where it was necessary to use them, bronze sliding plates were provided so as to reduce wear and corrosion as much as possible. The other reason was that the steel structures as a rule were covered with a concrete protection, and it was somewhat difficult under these circumstances to arrange a sliding joint which would have no water-pockets and which could be readily maintained.

The policy has been to cover steel structures with concrete or "gunite" in order to decrease the cost of maintenance. There were exceptions to this rule, however; for example, where through truss construction was used, the steel below the level of the deck was covered, while the part above was left uncovered and was painted. The principal exception, however, was

the high-level and long-span viaduct between Jersey City and Newark. For this structure the additional steel required for carrying the weight of the protection would have increased the cost so materially that it did not appear to be warranted economically.

Generally speaking, concrete covering was used on columns, brackets, fascia girders, and other places exposed to view, while "gunite" was used in less exposed places. The reason therefor was that, in the earlier stages of the work, the latter was difficult to obtain with sufficiently pleasing outlines to permit its use where exposed to view. In later years, however, the art of placing has improved considerably, and at the present time this type of covering can be obtained with sharp, straight lines and even surfaces.

Members that were to be encased in "gunite," were designed with cross-sections to which the covering could be readily and economically applied, and the results indicated that material savings in cost could be made by suitable design of the members to be covered.

The thickness was generally specified as $1\frac{1}{2}$ in., but this thickness can probably be materially decreased without impairing the value of the covering, and with considerable decrease in the cost, not only on account of the smaller amount of material used, but also on account of the smaller tonnage of steel required to carry the extra weight of this type of covering. Sufficient evidence has not as yet been gathered, however, to warrant a decrease materially below the standard thickness of $1\frac{1}{2}$ in.

The high-level viaduct structure between Jersey City and Newark, for which the span lengths varied from about 200 to 550 ft, was designed as a deck-truss structure, except at the river crossings where through-truss construction was used. The structure throughout is a series of cantilever spans with intervening suspended spans. The deck structure consists (*a*) of cross-girders set on the top chords of the main trusses at their panel points, which are normally 25 ft apart; (*b*) of stringers framed into the cross-girders about 8 ft apart; and (*c*) of secondary cross-beams supported on the stringers. This arrangement was made to permit the main reinforcement to be placed lengthwise of the structure rather than transversely. The secondary cross-beams were spaced at varying distances apart ranging, in a 25-ft panel, from 3 ft $6\frac{1}{2}$ in. at the ends to 4 ft $8\frac{1}{2}$ in. at the center of the panel, so as to equalize the moments in the reinforced concrete roadway slab, which rests directly on the secondary cross-beams.

In order to get the necessary under-clearance across the dredged channels of the two rivers crossed, without undue rise and fall of the highway profile, the channel spans were designed as through trusses and, for the purpose of obtaining harmonious lines, the flanking spans were carried gradually from the higher level of the through trusses to the lower level of the deck trusses, as shown in Fig. 7. As the through trusses are 57 ft, center to center, and the deck trusses are 36 ft, center to center, it was necessary to make a horizontal break in the truss line of the flanking spans at the point where they reach the level of the deck trusses. This break was quite simple in design, and is not noticeable when the structure is seen at a distance; but from a point on the ground near the side of the viaduct, looking lengthwise to the

point where the break occurs, it appears as if the curved lower chord is out of alignment. This condition was appreciated in making the design, but the only way that it might have been avoided was by tapering the trusses horizontally throughout the entire length of the flanking spans, which would have added materially to the cost of construction.

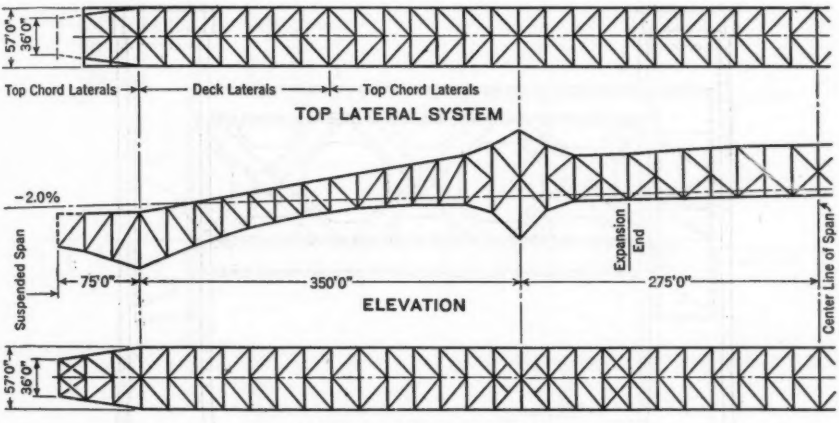

FIG. 7.—THROUGH TRUSS SYSTEM, FOR RIVER CROSSING

The trusses are supported on concrete piers by shoes of heavy steel castings with forged steel pins. On alternate piers the shoes are set on rollers to allow for changes in length due to changes in temperature.

The suspended spans are fastened at one end to the supporting cantilever arm by a hinged connection in the top chord. Actually, a simple riveted connection was used, because it was found that the deflection angle of the cantilever arm was practically the same as that of the suspended span. In other words, there is a point of contraflexure at the point of connection. The expansion end of the suspended span was supported by a vertical strut with pin connections at both ends. The reason for using a strut rather than a hanger was principally to maintain the same arrangement of diagonals throughout. If a hanger had been used, it would have been necessary to reverse the direction of the diagonal at this point.

The steel superstructure was designed to be supported on concrete piers. Each truss was supported on a separate shaft and, as there were normally two trusses to be supported, each pier bent had two pier shafts. At the location of the center ramps, however, where the structure was widened and was supported on three or four trusses, the number of pier shafts was increased accordingly. The shafts were designed with vertical outlines throughout their height, except near the bottom where they were flared out so as to form a base. Where the height was such as to require it, the shafts were connected with a horizontal strut, and, at greater heights, a truss was used instead of the strut. Both the truss and the strut were designed as steel members encased in concrete, but in order to conceal the steel outlines for the sake of appearance,

the concrete covering was made so heavy that when finished, they had the appearance of masonry rather than of steel covered with concrete. Fig. 8 shows one of these pier bents.

All viaduct structures were designed for an H-20 loading. For the high-level viaduct between Jersey City and Newark the specifications used were

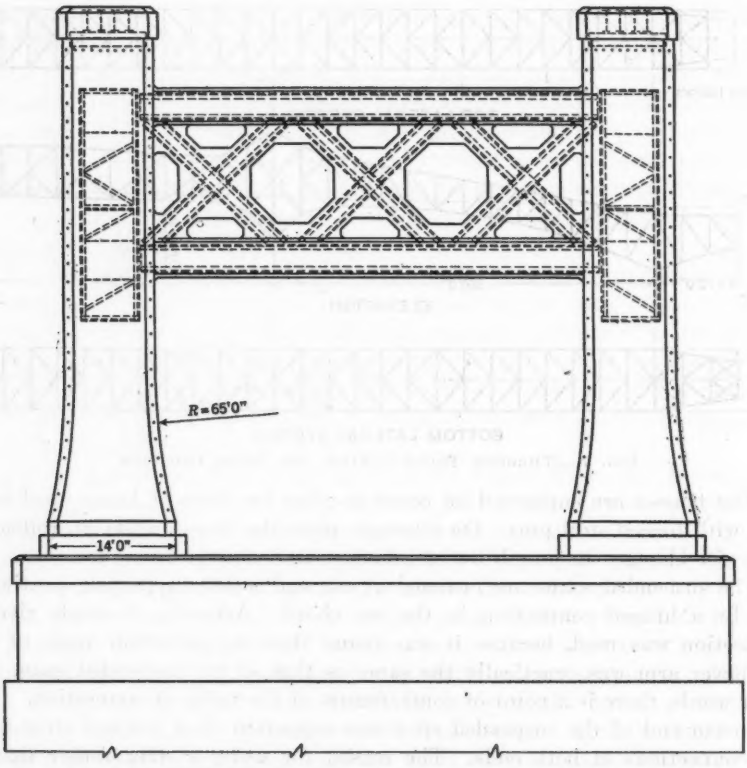

FIG. 8.—CROSS-SECTION OF PIER BENT, FOR STEEL SUPERSTRUCTURE

those of the American Association of State Highway Officials for Steel Bridges, which were published about the time this work was begun. The specifications used prior to that time were those of the American Railway Engineering Association.

EXPANSION JOINTS

For the high-level steel viaduct structure just described, the expansion joints in the main structure are normally about 600 ft apart, requiring provisions to be made at the joint for a movement of 6 in., or more. The type of expansion joints designed for the roadway slab is shown in Fig. 9, Type 1. It consists of two sets of steel castings provided with a series of parallel fingers, each about $1\frac{3}{4}$ in. wide, and deep enough to carry the load that may come upon them. One set of these "finger castings" is placed at the one side

of the expansion joint and the other set at the other side so that the fingers intermesh and provide a continuous surface at any position the structure may take due to expansion or contraction. The finger castings, which extend all the way across the roadway, were designed to be made in sections from

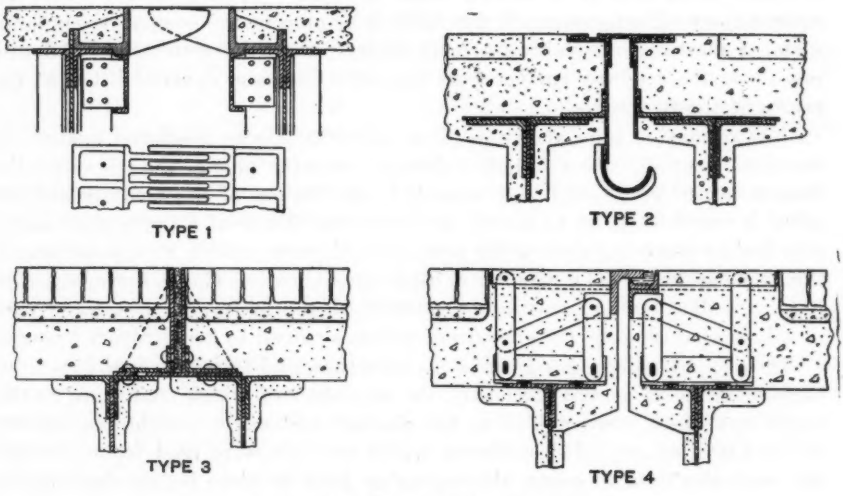

FIG. 9.—TYPES OF EXPANSION JOINTS

4 to 6 ft in length, and the openings between the teeth were about $\frac{1}{4}$ in. wider than the teeth so as to insure free movement of the joint. Similar castings were designed and used for the joints in the curb and sidewalk.

For viaduct structures of shorter span lengths, and where it was desired to provide tight joints, several types of expansion joints were used, but were changed in subsequent designs because the earlier designs did not prove to be as satisfactory as desired. The earliest type (Fig. 9, Type 2) was designed for the surface roadway over the subway in Jersey City. This type was found to be unsatisfactory for several reasons: First, the rivets in the checker-plate would wear loose, causing the plate to rattle in an undesirable manner, whenever a vehicle passed over it; and, furthermore, in certain cases, the entire expansion construction at one side of the joint would wear loose. This was due to the asphalt roadway surface wearing down adjacent to the concrete behind the steel expansion joint. The result was a constant pounding of the concrete by wheels striking against the exposed edge and forcing it against the steel of the expansion joint, which would gradually pulverize and destroy the concrete. Finally, the gutter between the twin girders would fill up with dust and dirt, and difficulty was experienced in removing this accumulation. It was possible for a man to enter the space between the two girders for the purpose of cleaning, but he could not make a visual inspection of it; all he could do was to feel his way along.

For subsequent work, therefore, the expansion joint, shown as Type 3 in Fig. 9, was designed. This type was found not to be entirely satisfactory

either, partly because of the difficulties of making an adequate enclosure at the ends of the joint, and partly because of the large volume changes that would occur in the space filled with mastic whenever the joint opened or closed due to temperature changes. If the joint opened, the mastic would drop below the roadway level, and if it closed, large quantities of the mastic would be squeezed out. Furthermore, it was difficult to prevent the comparatively large mass of mastic from seeking a level surface, with the result that it flowed out above the roadway surface near the curb lines and dropped below at the crown of the roadway.

The expansion joint shown as Type 4 in Fig. 9, was developed finally. It consists essentially of a small steel channel set at one side of the joint with the flanges turned up. One flange extends to the surface of the roadway and the other is cut down so as to permit the horizontal flange of a heavy steel angle, attached to the other side of the joint, to slide over it with its top surface at the roadway level. The channel is filled with mastic. The advantage is that only a small volume of mastic is needed to fill the joint, and the greater part of this is unaffected by the changes of volumes caused by temperature changes.

In order to anchor this joint to the structure, and to reinforce the roadway surface at the joint, flat steel bars, the top edges of which conformed to the roadway surface, were welded to the channel and angle. Light frameworks of flat bars were provided as shown, which not only were used for anchorage, but were also used to secure the expansion joint in place before the concrete was poured. This expansion joint has been in service for several years and has been found satisfactory.

Roadway Surfaces

At the beginning of the work the policy was established that the roadway surface should be paved with granite blocks, and this policy was followed for several years; but it was modified subsequently, and several other types of pavement were used on the highway. Surface roadways paved with granite blocks were designed to be laid on a concrete foundation. In the early designs, the foundation, due to the expected heavy truck loads, was made 10 in. thick, with top and bottom reinforcement. In later designs, this was changed to a concrete slab, 8 in. thick, without reinforcement, as this appeared to be sufficient where the sub-surface was firm.

The granite blocks, which have an average depth of 5 in., were set on a cement mortar cushion with a mastic filler in the joints. The surface was parabolic in cross-section, with a 6-in. crown for a 50-ft roadway. The mastic filler was not entirely satisfactory in all cases, because in hot weather it had a tendency to flow and cover the stone blocks, and eventually found its way to the low level of the roadway at the curb. For this reason later designs provided for a cement-mortar filler, except at intervals of about 30 ft, where three or four cross-joints, as well as the adjacent longitudinal joints, were filled with mastic. This type of construction has been used for only a comparatively short time, so that nothing definite can be said about it; thus far, however, it appears to be satisfactory.

In certain places where the highway occupies previously existing streets, the roadway was paved with a bituminous concrete surface on a concrete foundation. This was in deference to the wishes of authorities in the local municipalities; but it has a tendency to become wavy under the heavy traffic. It has the advantage, of course, that whenever openings must be made in it for construction or repairs of sub-surface utilities, it can be replaced without leaving an appreciable mark on the surface. On the other hand, as the work has been designed so as to keep sub-surface utilities outside the area of the paved highway, as far as possible, there should be little if any reason for making openings in the pavement.

A bituminous concrete pavement on a macadam foundation was found suitable as a temporary expedient when placed on fill that has not been entirely compacted. Of course, it does not remain smooth under heavy traffic, but it appears to stand up for a few years without undue maintenance. In some parts of the earlier work attempts were made to use surface-treated macadam for such temporary pavements, but it would not stand up under the heavy traffic.

A concrete pavement was adopted for the section adjacent to Newark Airport. The slab was made 10 in. thick, with top and bottom reinforcement. This heavy construction was used because there was some doubt whether the fill on which it was to be constructed, had finished settling. At other places, where the sub-surface conditions were more favorable, concrete pavements were designed with a thickness of 8 in. and a single top reinforcement. They were built with a V-crown and with a side slope of $1\frac{1}{2}$ in. in 10 ft. There has been a tendency to decrease the slope to about 1 in. in 10 ft.

Where granite block pavement was used on viaduct structures, the concrete floor-slab was water-proofed and the water-proofing was protected with a layer of mortar or concrete. The pavement was placed on this protective course in a manner similar to that used on surface roads.

An asphalt block pavement was adopted on a part of the highway viaduct in Jersey City, largely for the purpose of comparing this type with others. In this case, the concrete floor-slab was water-proofed in a manner similar to that specified for granite block pavements.

For the high-level viaduct between Jersey City and Newark, considerable study was give to the selection of the most suitable type of pavement. Granite block pavement was not favored on account of its heavy weight, and, for a time, it was not considered advisable to use a type of pavement for which the wearing surface was an integral part of the carrying slab. For that reason serious consideration was given to paving the roadway with some asphaltic surface, on top of the concrete floor-slab. In the meantime, several of the bridges built by the Port of New York Authority and other important bridges had been provided with a concrete surface integral with the floor-slab, and the results appeared to be quite satisfactory. Based on this evidence, as well as on that furnished by the many concrete roadway surfaces in the State of New Jersey and elsewhere, it was decided finally to use a concrete roadway surface integral with the concrete floor-slab.

While this study of pavement type was under way, the design and fabrication of the steel structure had progressed so far that it became necessary to establish a definite slab thickness, before the exact type of floor-slab had been decided upon, and, for that reason, a thickness of $8\frac{1}{2}$ in. was adopted as being that which might be used for a standard reinforced concrete slab.

Following this an intensive study was made of the various types of slabs that might be used, and several types were found suitable. The choice, therefore, depended on which would prove to be the most economical, and, for that reason, designs for four alternative types were prepared and submitted for bids.

Type 1 (Fig. 10), which was chosen eventually, is a concrete slab, $8\frac{1}{2}$ in. thick, reinforced with fabricated trusses, composed of round bars welded together. Essentially, it consists of a top and a bottom chord of round bars

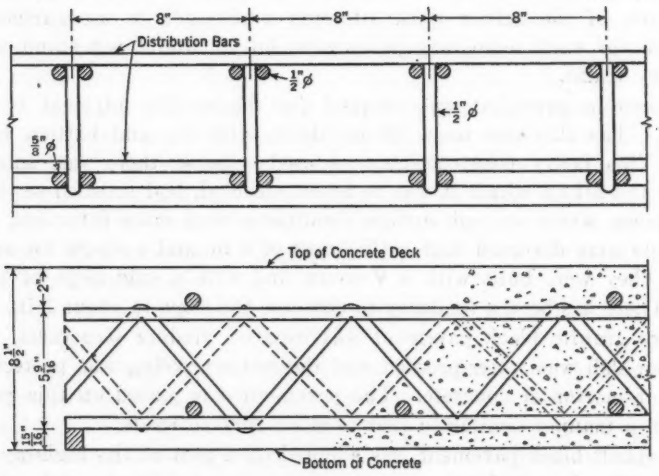

FIG. 10.—CROSS-SECTIONS OF TYPES OF FLOOR-SLABS FOR ROADWAY SURFACES

with a web member made of a bar bent so as to form the diagonals of the truss, the various pieces being connected by pressure welding. In some cases the trusses are fabribated with single bars for top and bottom chords and with the web member placed between the two chords, while in other cases each of the chord members consists of two bars welded to the sides of the web member at its panel points.

Type 2 was a series of inverted 5-in. T-beams, or half 10-in. H-beams, set closely together and tack welded so as to form a solid steel surface at the bottom of the slab, and filled with concrete to a height of 2 in. above the top of the T-beams. This type of slab reinforcement is usually fabricated in depths ranging from $2\frac{1}{2}$ to $3\frac{1}{2}$ in. The 5-in. depth was required because it was thought that while the slab of a lesser depth might be designed to carry the required load, it was not entirely certain that the concrete in it would not be crushed, in time, by constant use.

Type 3 was a solid-bottom plate with ribs at intervals and a steel grating attached to them with its top surface at a height of 5 in. from the bottom of the plate; concrete was placed on top of the plate to a depth of 2 in. above the steel. Type 4 was a solid-bottom plate with welded trusses, 5 in. deep, attached as ribs, and concrete was placed to a depth of 2 in. above the top of the ribs.

In all four cases the reinforcement also involved transverse top bars secured in place in various ways and, in addition, transverse bottom bars were required for Type 1. Each type was carefully investigated and found to be suitable for the purpose. For Types 2, 3, and 4, however (which had an over-all depth of 7 in.), it was necessary to introduce fillers, $1\frac{1}{2}$ in. deep, between the steel support and the floor-slab, in order to make them conform to the depth of slab for which the structure had been designed. This, naturally, added to the cost of these three types although, on the other hand, the volume of concrete required was less.

The trusses in Type 1 were designed to be spaced on 8-in. centers and to be 5 in. deep over all. The two top bars and the web member were $\frac{1}{2}$ in., and the two bottom bars, $\frac{5}{8}$ in., in diameter. Rectangular spacing bars, as shown, were welded to the cross-beams on which the trusses were supported, and round top and bottom distribution bars, extending the full width of the roadway, were provided, as shown. In order to facilitate the work of erection and to insure exact spacing of the reinforcement, it was required that the trusses should be assembled in mats prior to erection by welding them together by the spacing and distribution bars. The length of the mats was required to conform to the distance between the primary floor-beams, which distance normally was 25 ft. The width was not specified, but left to the option of the Contractor, subject to the approval of the Engineer. The width of the mats actually furnished by the Contractor was normally 10 ft, but it is probable that the erection would have been easier if the width had been about one-half that actually used. The reinforcement trusses were arranged to be placed longitudinally on the structure, because better wheel-load distribution could be obtained in this manner than with a transverse arrangement of the trusses.

The concrete for the slab was required to have a crushing strength of not less than 4 000 lb per sq. in., when 28 days old. Actually, the concrete furnished had an average strength of about 4 800 lb. In computing the strength of the slab, the upper 2 in. of the concrete was not taken into account. When the time comes that it may be necessary to renew the wearing surface of the roadway it will thus be possible to remove this top layer without affecting the carrying capacity of the slab, and in this way a new wearing surface can be placed. It was thought that this new surface might be either concrete, or some type of asphaltic surface—or possibly some entirely new type of surface pavement which has not as yet been developed.

Curbs, Sidewalks, and Railings

No provisions have been made on any of the viaduct structures for sidewalks for pedestrians. Provisions have been made, however, for footwalks

about 2 ft wide on both sides of the roadway, to be used in connection with the policing and maintenance of the highway and for emergency purposes. Except on the high-level viaduct the curb height of these footwalks is normally 8 in. above the roadway.

The general details of the footwalk as used on all steel structures, (except on the high-level viaduct between Jersey City and Newark) are shown in Fig. 11. In the earlier designs no provisions were made for any support of the curb angle during construction, but it was found that the contractors

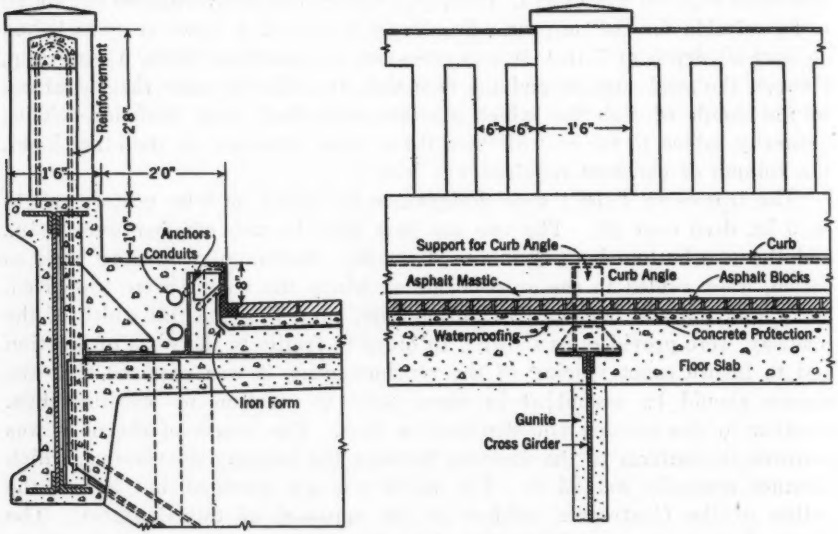

FIG. 11.—DETAILS OF FOOTWALK USED ON STEEL STRUCTURES

had some difficulties in aligning the curb angles properly and in securing them during the pouring of the concrete. In later designs, therefore, steel supports were provided, as shown in Fig. 11, which furnished a direct connection between the curb angle and the steel structure. These supports materially facilitated the proper erection of the curb angles.

For further facility of erection the design provided for the placing of a sheet-metal form reaching from below the curb angle to the top of the concrete roadway slab. In conjunction with the curb angle, this metal form usually made it possible to pour the concrete of the roadway and footwalk slab without additional form work, and further provided a means of flashing the roadway water-proofing against the face of the curb.

The railings on all viaducts were made of concrete as shown in Fig. 11. On account of the narrow footwalk, it was considered essential to provide a railing strong enough to withstand the impact of any vehicle that might accidentally mount the curb and strike against it. Therefore, it was made quite substantial, and every one of the vertical posts was securely anchored against overturning.

On the high-level viaduct it was appreciated that special precautions should be taken, not only to prevent vehicles from falling from the viaduct, but also to allay the fear of such nervous drivers as are not able to look down from great heights without a qualm. These considerations were particularly important on this structure because, due to the narrow footwalk, the roadway is quite close to the side of the structure.

For these reasons, the curb height was made 16 in. above the roadway surface as shown in Fig. 11, so as to make it practically impossible for a vehicle to mount the footwalk. Behind the curb and forming the footwalk a horizontal plate girder was attached to the main steel structure at each panel point of the supporting trusses and covered on top with concrete. The vertical face of the curb consisted of a steel plate attached to the flange of the horizontal girder, and this plate extended to within 5 in. of the roadway surface, leaving an opening to permit drainage over its edge.

The top of the railing was made 4 ft 5 in. above the footwalk level, or 5 ft 5 in. above the roadway level at the curb. With this height it is not possible for a driver of a passenger car to look over the railing to the meadows below and, if he is properly attending to his driving, he cannot look through the railing between the posts; in fact, the railing appears to him as a solid wall. On the other hand, if the car is moving at a reasonably fast rate of speed, the passengers in the car, who are able to look squarely at the railing, will be able to look through the railing as if there were no posts at all.

Drainage

The profile of the highway has generally been laid on a minimum gradient of 0.5%, so as to provide for proper drainage of the roadway. In undeveloped territory the cross-section was designed so as to permit the water to flow over the sides and, when necessary, side ditches were provided to carry the water to existing drainage channels. In due time, however, it may be necessary to provide curbs, and storm sewers must be built at that time. Preliminary plans of the future sewer were prepared, therefore, and the cross-drains necessary according to these plans were constructed prior to the permanent paving of the roadways.

Where the highway was constructed on embankment with curbed roadways, a complete drainage system was provided and connected either to the municipal sewer systems or to specially constructed drainage systems. Frequently, in the earlier designs, a 12-in. pipe was used, but experience indicated that this size was rather too small, and during the later years the minimum diameter was increased to 18 in.

The catch-basin heads, used on curbed surface roads, have a curb inlet and a grating that is slightly dished for the purpose of collecting the water more effectively. This is the standard design of the Highway Department, but it might be better to make the gratings flat, because drivers of cars have the tendency to avoid driving over the dished gratings, with the result that 2 or 3 ft of the width of the roadway is wasted. On a viaduct, catch-basins of a similar type, but without curb inlet, were generally used.

The basins are connected with 8-in. cast-iron pipes attached to the columns of the structure, which carry the drainage to the sub-surface sewer system.

In order to carry these pipes from the catch-basin down the structure and past the column base it was frequently necessary to introduce several bends or other fittings in the run. As each fitting appears to be a potential location for stoppage of the flow of drainage water, and as each additional fitting rapidly increases the difficulty of cleaning the drain, the writer has considered in future work, where conditions will permit such an arrangement, to place the catch-basin some distance away from the column bent and to carry the pipe vertically down to the ground level. Of course, this would necessitate a special support for the pipe, but it would facilitate the maintenance of the drains materially.

In the subway, the drainage was extended to two longitudinal drains built under the sidewalks as shown in Fig. 6. These drains divert the water to the ends of the subway, where it emerges in the open on the slopes of the hill through which the subway section was built, and from these points the drainage water flows through pipes to the existing sewer systems.

Organization

The work of which the design has been described, was carried out under the direction of the New Jersey State Highway Commission and the State Highway Engineer. W. G. Sloan, M. Am. Soc. C. E., was the State Highway Engineer from the inception of the work until 1929, when he was succeeded by J. L. Bauer, M. Am. Soc. C. E. Fred Lavis, M. Am. Soc. C. E., was in charge of the work from the beginning until 1928, when he was succeeded by H. W. Hudson, M. Am. Soc. C. E. The writer was Designing Engineer from the inception of the work to its completion.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

PRACTICAL RIVER LABORATORY HYDRAULICS

BY HERBERT D. VOGEL[1], ASSOC. M. AM. SOC. C. E.

SYNOPSIS

The science of laboratory hydraulics as applied to determinations of problems relating to the control of rivers is of only recent origin, although hydraulic experiments of other kinds have been common for many years. Since the beginning of the Twentieth Century, however, considerable progress has been made, and while much still remains to be learned of limiting laws and conditions, it appears, nevertheless, that the time has come to take stock of the practical information available.

It is the purpose of this paper to summarize, as briefly as possible, existing data and to describe proven methods applicable to the solution of river hydraulics problems in the laboratory. By so doing, it should serve as a basis of procedure for the laboratory technician and as a guide to the field engineer who wishes to know what specific types of problems may be reasonably subjected to laboratory treatment.

INTRODUCTION

The utilization of models as a means for the determinnation of full-scale performances was conceived nearly 250 years ago when Sir Isaac Newton in his monumental treatise, the "Principia," stated in a single paragraph the Principle of Similitude. For more than 175 years this idea lay dormant; then it was seized upon by Froude, an English Naval Engineer, who applied it to studies aimed to ascertain the best design for a ship's hull. Numerous models were constructed by Froude, and in a narrow, water-filled tank they were towed at varying velocities, while accurate measurements were made with a view to determining the tractive resistances encountered. These experiments with moving objects in still water quickly suggested the possibility of investigating, through the use of small-scale replicas, the effects of flowing water on stationary objects, and, similarly, the effects of engineering structures on the regimen of rivers.

NOTE.—Discussion on this paper will be closed in February, 1934, *Proceedings*.

[1] 1st Lieut., Corps of Engrs., U. S. A.; Director, U. S. Waterways Experiment Station, Vicksburg, Miss.

In 1875, the first true river models were constructed in France by Fargue, who attempted to determine effects resulting in prototypes a hundred times as large as the miniatures with which he dealt. His models were of the Garonne River and his problem was to determine methods of improving the channel of that stream for navigation. Ten years after the experiments by Fargue, Professor Osborne Reynolds, of the University of Manchester, Manchester, England, invoked the Principle of Similitude by utilizing scale models in the investigations of flow in a tidal estuary of the River Mersey, at Liverpool. About a year later (1886), Sir William Vernon-Harcourt applied these principles in experiments for the improvement of navigation near the mouth of the River Seine in France, but his investigations, like those of his predecessors, were conducted haphazardly, without suitable equipment and without a thorough understanding of the basic principles of model research.

A renaissance era in the science of river research was initiated by Professor Hubert Engels, of the Technical University at Dresden, Germany, at the beginning of the Twentieth Century as the result of a visit to America a few years earlier. Professor Engels began his investigational work in 1891, but at that time possessed no laboratory and no organized staff for research. In the University of Michigan, at Ann Arbor, Mich., he found an embryo hydraulic laboratory with demonstration equipment then used for classroom instruction, which so inspired him that he returned to his own University with definite plans in mind for a permanent and well-ordered hydraulic research institution. His first laboratory was a modest affair installed in the basement of the University in 1898. Here, he gradually increased his range of experiments and a few years later rebuilt his laboratory in larger form. The utility of the methods which he introduced quickly attracted the attention of other German engineering colleges, where new and larger laboratories were built—guided often by the friendly personal counsel of Professor Engels.

In 1903 there appeared on the horizon a new personality destined to influence largely the development of hydraulic laboratory methods and the promulgation of the laboratory idea. This was Dr. G. de Thierry, who, in that year, received his appointment as Professor of Canal and Harbor Engineering and of Hydraulics at the Technical University in Berlin, Germany. Professor de Thierry founded an efficient student laboratory at his institution of learning and, assisted by Professor Ludin, of the same University, did much toward promoting the development of the Experiment Institute for Hydraulics and Naval Architecture in Berlin. This Institute built in 1902 and 1903 by the Prussian Government, carries definitely associated with it the name of Hans Detlef Krey, who, until his death in 1928, stood pre-eminent in the field of practical hydrodynamics. Under the supervision of Professor de Thierry and the sponsorship of the late John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., the monumental treatise, "Wasserbaulaboratorien Europas," came into being in 1926 as a summary by leading directors of the accomplishments of their respective laboratories. The

same book translated into English and published in 1929 under the title, "Hydraulic Laboratory Practice," stands to-day as a leading reference in the subject of hydraulic research. Because of the comprehensive manner in which this work has treated the subject it is deemed unnecessary to elaborate here on the details of the many laboratories now existent in the world as offspring of the parent institution founded by Professor Engels. Suffice it to say that the idea has been received with considerable favor by engineers and that the mathematical doctrine stated somewhat obscurely by Newton, in a single paragraph, has been expanded and elaborated to cover a wide range of problems, from turbine design to the improvement of harbors. Researches carried on by means of the principle relate to: (*a*) The improvement of both the efficiency and capacity of hydraulic turbines; (*b*) the finding of remedies for the evils of cavitation in turbines; (*c*) improvements in efficiency and capacity of centrifugal pumps and propeller pumps; (*d*) the improvement of the designs of spillways for water-power dams; and (*e*) the designing of great sluice-ways so as to present a minimum loss of head. Another important problem is that of finding the best means of dissipating energy of flood waters flowing over a high dam where it is tending to cause damaging scour in the river bed below the dam.

The scope of hydraulic laboratory practice has been extended since about 1918 to include studies of bed movement in rivers, and methods of changing or influencing such movement; to investigations aimed at the determination of effects of cut-offs in large alluvial rivers; to experiments dealing with means of preventing damage to railway embankments during overflow by flood waters; and to hundreds of similar studies and investigations. In spite of this rapid expansion of the utility of the hydraulic laboratory, it appears probable that its greatest service to hydraulic science remains yet unperformed. The practical field engineer has had a new and most important tool placed at his disposal, but until he has made further advances toward mastering the technique of this science and adapting it to his purposes, it will not fulfill its full potential value. For this reason it is of the utmost importance that field forces be encouraged to maintain constant contact with tests in progress at the laboratory and that they be trained to observe, for themselves, the model phenomena and their interpretation in terms of Nature. Such observations may be of much greater benefit than a formal written report of the tests.

Hydraulic Similitude

General.—If two geometric figures are drawn so that all corresponding lines are of the same ratio and their corresponding angles are of equal degree, it will be found that their areas bear a fixed relationship, one to the other, that may be expressed by the square of the basic linear ratio. The two figures are then said to be similar, and from the one may be determined all the properties of the other. As defined by the dictionary, for a geometric sense, similitude is "the relation of identity between two figures irrespective of magnitude."

From an hydraulic standpoint, strict geometric similarity may often be subordinated to the necessity of preserving other, more important, relationships, and hydraulic similitude has been defined as referring to those "conditions which must be observed in experiments with a model in order to reproduce faithfully the phenomena occurring in full-scale performance." Where then, in such cases, a sacrifice of true geometric similarity becomes necessary it is on the basis of preserving dynamic or kinematic similarity as a means toward obtaining hydraulic similitude.

True geometric similarity implies undistorted linear relationships; that is, it requires that all corresponding linear dimensions in the full-scale and model must be in the same ratio. More generally speaking, however, it may be said that a kind of form similarity exists even in cases where the model is distorted, provided the distortion is a result of some definite and consistent plan.

Dynamic similarity requires that at corresponding times in full-scale and model the accelerations of corresponding mass elements must be in constant ratio to each other, and must be directed in the same relative directions. Since it is physically impossible to modify the effect of gravity in a model, the same reduction of accelerations, according to a fixed ratio, will fail to satisfy completely the foregoing requirements of dynamic similarity, but from the relationship existing between force and the product of mass and acceleration, it is evident that with corresponding forces bearing a constant ratio to each other, such similarity may be attained by making corresponding accelerations equal in model and prototype. Briefly stated, therefore, dynamic similarity is governed by the requirement that at corresponding times any two corresponding forces in full-scale and model must be in the same ratio, and to obtain this condition the conditions for mechanical and kinematic similarity must also be satisfied. The first of these requires corresponding weights in the two systems to possess a definite fixed ratio one to the other; the latter demands that the full-scale and model must be geometrically similar in all respects, at corresponding times; or, in the case of hydraulic models, the elements of flow must simulate corresponding elements in Nature with respect to both direction and time.[2] Difficulties naturally arise in the interpretation of results obtained from model studies because it is almost impossible to simulate all conditions and properties of Nature. Theoretically, this might be done in certain special cases if it were possible to control independently, and at will, all the properties of the materials in the model as well as the forces involved. These properties include the density, viscosity, and surface tension of the fluid used in the model; and the size, shape, cohesion, and density of the solid materials. The external forces include gravity, friction, and barometric pressure.

Scale Relationships and Nomenclature.—Although a number of papers have been presented on the subject of similitude, it is extremely difficult in

[2] See "Die Grundlagen der Aehnlichkeitsmechanik und ihre Verwertung bei Modellversuchen, unter besonderer Berücksichtigung schiffbau technischer Anwendungen," von M. Weber, Charlottenburg, Germany, 1918. For a slightly different conception refer to "Strömungslehre der Rohre und Offene Gerinne," von Schiller und Eisner, Leipzig, 1932.

many cases to follow the reasoning involved, due to complex systems of nomenclature. These systems are obviously the result of haphazard growth rather than predetermined plan, and English, German, and Greek letters will be found indiscriminately mixed.

At the United States Waterways Experiment Station, at Vicksburg, Miss., capital letters with the subscripts, m and n, are taken to indicate corresponding quantities in the model and in Nature. The ratio of these quantities expressed in the order, model to Nature, is denoted by the same letter, uncapitalized. Thus, the length scale is, $l = \frac{L_m}{L_n}$, and is always a fraction except in the case of the slope scale, which is an integral. This is in accordance with conventional designations and the usual understanding of scale values pertinent to other sciences. The scale of a map, for example, is conventionally expressed as 1:25,000 rather than 25,000 which, by itself, constitutes only a meaningless number. The following letters are taken to represent commonly used hydraulic elements:

A = area of cross-section.
B = a distribution factor pertaining to relative fineness of sand.
C = a roughness coefficient (see, also, N).
D = depth, or head; also, diameter.
E = diameter of sand grains.
F = critical tractive force per unit area.
G = acceleration of gravity.
H = head (see, also, D).
J = weight of fluid per unit of volume.
K = a constant for practical purposes (in Equation (10), $K = \frac{Q}{L D^{\frac{3}{2}}}$; also, coefficient of kinematic viscosity.
L = length, or breadth.
M = modulus of viscosity.
N = coefficient of roughness.
P = wetted perimeter.
Q = discharge.
R = hydraulic radius.
$\mathbf{R}$ = Reynolds' number.
S = slope.
T = time.
V = velocity.
Z = turbulence criterion.
a = a basic scale ratio.
a = a scale ratio of areas, $\frac{A_m}{A_n}$.
d = a scale depth, $= \frac{D_m}{D_n}$.
e = a scale ratio of sand-grain diameters, $\frac{E_m}{E_n}$.

j = specific gravity of water; j_1 = specific gravity of bed material; also, j = scale ratio, $\frac{J_m}{J_n}$.

l = a scale length = $\frac{L_m}{L_n}$.

m = a subscript denoting a model characteristic.

n = a scale ratio of roughness coefficient; also, a subscript denoting a prototype characteristic (n, for Nature).

p = a scale ratio of the wetted perimeters, $\frac{P_m}{P_n}$; also, density.

q = a scale discharge, $\frac{Q_m}{Q_n}$.

r = a scale ratio of hydraulic radius, $\frac{a}{p}$.

s = a scale slope = $\frac{S_m}{S_n} = \frac{D_m}{D_n} \div \frac{L_m}{L_n} = \frac{d}{l}$.

t = a scale ratio of times, $\frac{T_m}{T_n}$.

u = viscosity coefficient.

v = a scale velocity, $\frac{V_m}{V_n}$.

This system is designed to apply particularly to practical hydraulic research and is not essential to the case of ideal similarity, such as will now be discussed.

Similarity of perfect fluids requires only a consideration of the force of gravity and, therefore, all others, such as friction and viscosity, may be neglected. When this is true, all involved quantities may be expressed as simple functions of the basic scale ratio, which will be denoted as α. The following list is based on such a conception:

Quantities	Scale values
Angular accelerations	$\alpha^{-1.0}$
Revolutions per unit of time	$\alpha^{-0.5}$
Accelerations, constants	$\alpha^{0.0}$
Velocity and time	$\alpha^{0.5}$
Linear dimensions	$\alpha^{1.0}$
Discharge per unit time per unit width	$\alpha^{1.5}$
Areas	$\alpha^{2.0}$
Discharge per unit time	$\alpha^{2.5}$
Volumes, masses, forces, weights	$\alpha^{3.0}$
Power, momentum, impulse	$\alpha^{3.5}$
Work, energy, moments	$\alpha^{4.0}$

The values in this tabulation are directly obtainable from the formula for velocity produced in a freely falling body by the influence of gravity,

($V = \sqrt{2GH}$). Beginning with this relationship:

$$\frac{V_m}{V_n} = \frac{\sqrt{2GH_m}}{\sqrt{2GH_n}} = \frac{\sqrt{2GD_m}}{\sqrt{2GD_n}} = \frac{\sqrt{D_m}}{\sqrt{D_n}} \quad \ldots\ldots\ldots\ldots (1)$$

Whence,

$$v = \sqrt{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

or, expressed in words, the velocity scale is equal to the square root of the vertical scale. This relationship, together with that for the time scale, $t = \sqrt{d}$, is known as Froude's law and is the basic equation for the determination of velocity scales in hydraulics and other sciences, in which gravity is the controlling influence. It should be noted, however, that Equation (2) is for the effects of gravity only and is not applicable to conditions involving frictional or other coefficients. From it is derived the discharge scale:

$$q = l\,d^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

obtained by multiplying scale values for area and velocity.

In case the model is designed to show the effects of discharge over dikes, dams, or other structures functioning primarily by gravity, it is obvious that only the relationship expressed by Equation (3), is directly applicable to an expression of the natural phenomenon involved. In the case of a weir this relationship may be determined in another way, by applying the principles of similitude to standard weir formulas of the form,

$$Q = KLD^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Factor K in this case disappearing as a constant in the resulting formula for similitude. For practical purposes, K may be assumed. Actually, experiments indicate that K is a function of D to a certain extent. Here is a first indication of the complexities that may be encountered in attempting to simulate, completely, the effects of both main forces (gravity and friction) in connection with any hydraulic model study. Fortunately, however (and generally), in practice, one of the forces may be eliminated from consideration, as in the present instance. In so far as K is not a constant, but presents frictional characteristics varying with D, a slight error will be introduced by Equation (4), which was derived solely from a gravitational relationship.

The validity of the Froudian relationship, even in cases where the frictional variances may be fairly large, is indicated by the results of experiments with full-scale and small-scale railroad embankments at the U. S. Waterways Experiment Station. These tests were conducted to determine the erosive action and general destructive effects of flood waters, to devise methods of protecting the embankment, economically, against such effects; but since several models were used they served nearly equally well to show existing scale relationships. Table 1 of readings at corresponding times, illustrates the observed similarity.

Open-Channel Models, Undistorted.—As previously stated, fulfillment of the conditions establishing perfect similarity is seldom had in connection with problems pertinent to flowing water, and except where gravity might

TABLE 1.—SCALE RELATIONSHIPS BETWEEN MODEL AND FULL-SCALE EMBANKMENTS

Item No. (1)	Description (2)	Full-scale embankment (3)	1:5 scale embankment (4)	1:5-model results transferred to full scale (5)
	Gauge Reading 30 Ft (Distance in Full Scale) from Center Line of Track:			
1....	(a) Up stream	3.0	0.60	3.0
2....		3.2	0.63	3.2
3....	(b) Down stream	2.4	0.48	2.4
4....		2.8	0.59	3.0
5....	Velocity, in feet per second	5.4	2.6	5.8
6....		5.2	2.4	5.4

be assumed to act alone, as in the case of flow over dams, weirs, etc., modifications must be made according to the nature of the predominant forces.

Where a movement of water occurs in relatively shallow channels, as is frequently the case in Nature, the factor of bed roughness assumes such proportions as to make friction practically a controlling consideration, and no longer can the relations, previously cited, be adhered to unless secondary factors are introduced for their modification. Rather than attempt this, however, it is often better to determine scale values from well-known hydraulic formulas, such as those developed by Kutter and Ganguillet, Bazin, and Manning. Making use of the formula of the latter:

$$Q = A\,V = \left(\frac{1.486\,A}{N}\right) R^{\frac{2}{3}}\,S^{\frac{1}{2}}$$

whence,

$$q = \frac{Q_m}{Q_n} = \frac{A_m}{A_n}\left(\frac{N_n}{N_m}\right)\left(\frac{(R^{\frac{2}{3}}_m\,S^{\frac{1}{2}}_m)}{R^{\frac{2}{3}}_n\,S^{\frac{1}{2}}_n}\right) = \frac{a\,r^{\frac{2}{3}}\,s^{\frac{1}{2}}}{n} = \frac{a^{\frac{5}{3}}\,s^{\frac{1}{2}}}{n\,p^{\frac{2}{3}}}$$

By correcting the finally deduced relationship for frictional variations in channels of varying shapes, the same is then directly applicable to problems involving distorted scales.

If no distortion is involved, horizontal and vertical scales will be equal, and the formula for discharge will be reduced to $q = \frac{l^{\frac{10}{3}}}{n\,p^{\frac{2}{3}}}$, or, assuming the hydraulic radius equal to the mean depth,

$$q = \frac{l^3}{n} \qquad (5)$$

Since the velocity scale, $(v) = \frac{q}{a} = \frac{q}{l^2}$,

$$v = \frac{l^3}{n} \qquad (6)$$

Other functions of the model may be determined in the same manner on a basis of the Manning formula or, if desired, from the Chezy or other relationships.

Although many European experimenters have expressed themselves unmistakably as opposed to the use of distorted models, nevertheless it is impossible in many cases to adhere to true similarity of reproduction on a small scale. As a matter of fact, there is scarcely a conceivable problem with respect to the Mississippi, or any other alluvial river, for which a solution can be obtained without resort to distortion because, where depths bear small ratios to widths, a reduction of linear elements by the same scale results in shallows in the model that are similar in their functioning (through the phenomena of surface tension and adhesion) to nothing that exists in Nature. Moreover, the velocity as deduced by Froude's law may be so small as to cause stream-line flow in the model, whereas flow in Nature is almost invariably turbulent. In turbulent flow the surface slope of the fluid (and, therefore, the friction head) increases in proportion to the square of the mean velocity, while in stream-line flow it increases only as the first power of the velocity. It is plain, therefore, that the characteristics of flow will be entirely different under the two conditions, and it becomes necessary to provide, in each case, that natural phenomena are accurately simulated.

Much has been said about geometric distortion in models and little about the distortion of other properties which occurs when the scale is reduced even if the model is not distorted geometrically. In reality, every model is distorted in some respects. The investigator must not be misled into overlooking, complacently, the less obvious distortions of properties and forces while he concentrates on avoiding geometrical distortion. The question is not whether distorted models will be used; it is rather, how can the distortion that is certain to occur in every model be made least harmful?

Referring once more to the Manning formula a relationship for similarity of discharge may be derived for distorted models in the same way as previously indicated. By dropping constants and substituting scale values for the hydraulic radius, R, slope, S, area, A, and the coefficient, N, there is evolved, as an expression of the discharge scale:

$$q = \frac{r^{\frac{2}{3}}\, s^{\frac{1}{2}}\, a}{n} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

If the model has been built according to principles of simple distortion—that is, with no artificial adjustment of slope—and if the cross-sections are flat enough to permit substitution of mean depth, d, for r, it is found that,

$$q = \frac{l^{\frac{5}{2}}\, d^{\frac{1}{2}}}{n} \text{ (approximately)} \ldots\ldots\ldots\ldots\ldots (8)$$

Where an additional slope is given to the model to facilitate bed-load movement, the equation becomes:

$$q = \frac{l d^{\frac{3}{2}}\, s^{\frac{1}{2}}}{n} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

These relationships are of value mainly as means of determining before the construction of the model, the quantity of water it will be necessary to supply. In an active laboratory with many models in operation it is frequently necessary to conserve the water or to regulate its distribution so that each model will be adequately served. It is essential, therefore, to know in advance approximately what discharge will be required. After the model has been put into operation, exact scales for discharge, velocity, and time may be determined experimentally in a manner to be described subsequently. To use Equation (8), however, even as a basis for estimates and design, an approximate evaluation must be made of the ratio, n, which is never accurately determinable. Fortunately, it has been found by experiment that for small-scale models (those in which $l < \frac{1}{1\,000}$), built with cement mortar, and for models of intermediate scale (those in which $\frac{1}{1\,000} > l > \frac{1}{500}$), built with a sand bed, the value of n is nearly unity. Since this n is the ratio between N of the natural river and the N of the model, there would appear at first to be an anomaly, because standard tables place the value of N in a reasonably good stretch of river at about 0.030 and at about one-half that in a concrete flume. The controlling factors of slope and hydraulic radius must be taken into consideration, however, and even casual thought will reveal that in a model river of small cross-section much resistance will be encountered even when the surface is relatively smooth.

Under the assumption that N may be taken as equal to unity, the following formula is obtained for discharge in the model of a river:

$$q = l^{\frac{1}{2}} d^{\frac{1}{2}} \quad \ldots\ldots\ldots\

determinations for channel flow, but are considerably at variance with the determinations by the gravity formula.

The velocity scale is obtained as previously indicated, of course, by dividing the scale for discharge by the scale for area, whence:

$$v = l^{-\frac{1}{2}} d^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

TABLE 2.—COMPARISON OF THEORETICAL AND ACTUAL DISCHARGE
(Data Obtained from Tests Conducted at the U. S. Waterways Experiment Station)

Scale ratios	Small-Scale, Concrete Surface				Sand Bed	
	A	*B*	*C*	*D*	*E*	*F*
$l =$	1:4 800	1:4 800	1:2 400	1:1 200	1:720	1:720
$d =$	1:360	1:200	1:120	1:48	1:72	1:72
$q = l^{\frac{1}{2}} d^{\frac{1}{2}}$	1:24 000 000	1:6 700 000	1:1 567 000	1:152 000	1:284 000	1:284 000
$q = l\, d^{\frac{1}{2}}$	1:32 800 000	1:13 600 000	1:3 155 000	1:399 000	1:440 000	1:440 000
Measured $q =$	1:21 000 000	1:7 200 000	1:1 570 000	1:150 000	1:300 000	1:280 000

The time scale as referred to hydraulic discharge is determined from the relationship: $\text{Discharge} = \frac{\text{Volume}}{\text{Unit time}}$. Whence, the time scale equals the volume scale divided by the discharge scale, or,

$$t = \frac{l^2 d}{q} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

This relationship is particularly applicable to reservoir problems, or to flood-control problems where the time element is important, such as in the filling of back-water basins and other areas of storage. It is not applicable to problems of channel contraction, sedimentation, or similar investigations involving rates of bed development. Such rates must be determined by actual tests conducted in Nature and in the laboratory. There is no known "short-cut" to such determinations, and formulas taking into account the transporting power of flowing water and the resistance to motion of grain particles in suspension or otherwise, are almost invariably misleading.

SIMILITUDE AS APPLIED TO BED MOVEMENT IN OPEN-CHANNEL MODELS

General.—In the preceding pages it has been shown that problems encountered in a river hydraulics laboratory fall naturally into two general groups. In the case of spillways, submerged dikes, or weirs, for which it is desired to determine coefficients of discharge, head over the structure, or other similar data, gravity may be considered as the controlling force; but in the case of open-channel models, bed and wall friction becomes of increasing importance. A compromise is frequently necessary in instances where dikes must be considered as a means of channel improvement, and seldom is it possible to apply any one set of laws indiscriminately to such a problem. Where the investigation is designed for the determination of back-water

limits, or the drawdown resulting from the removal of some obstacle resisting the flow of the stream, it may be that a fixed-bed model will suffice, but all too frequently it is necessary to build the replica with a bed of movable material that will be acted upon by various existent currents. Experiments with movable-bed models being of the most difficult type to perform, it follows that investigations designed to determine effects of dikes on river beds must be approached with caution and understanding.

In choosing a sand for the bed of the model it must be conceded that close adherence to principles of geometric similitude is entirely impossible because, assuming the model to be built to a scale of 1:500, the same particles of Nature would be so reduced in size as to present a microscopic appearance. The impalpable powder could not be used in the model under any operating condition because, due to its extreme fineness, it might either float, be transported in suspension as sedimentary matter, or become compacted so as to resist, completely, the tractive power of the model stream. Fine, uniform sands have been used throughout many experiments, but a cardinal objection is that invariably riffles of the bed form. These riffles, of course, bear a certain similarity to the sand waves of Nature, but their height may be so grossly exaggerated as to detract from the accuracy of the model results. In practice, the difficulty may be partly rectified by smoothing out, in the model, the ranges on which soundings are taken.

Certain limiting conditions must always be considered whenever it is planned to experiment with a movable-bed model. It is obviously impossible to simulate all physical properties of a natural stream entirely. Distortion of linear dimensions is sometimes necessary to produce measurable depths and proper turbulence. Excessive distortion may result in certain types of models, in an extreme and undesirable dissimilarity of flow phenomena. In addition to the factors that influence bed development (such as discharge, slope, etc.), there is also to be considered the guiding of the transported bed load. Preservation of geometric similarity, on the other hand, is practically impossible for the reasons given, namely, that a scale reproduction of the transported bed material would result in a fine material, subject to excessive riffling should any movement whatever occur. Just as it is often necessary to depart from conditions of true geometric similarity in the design of the model in order to keep it within workable limits and, at the same time, to obtain measurable depths and the proper character of flow, it is also necessary to exaggerate the size of sand grains, and, concurrently, it is frequently advisable to exaggerate the model slope in order to obtain similarity of the bed-load movement.

In departing from the laws of geometric similarity by distorting the model and utilizing a coarser grained bed material, thought must be given to those natural laws that are regarded as *a priori* principles of stream-bed development; that is, the plan of the stream must be unchanged from that of Nature and all curvatures must be reproduced in their exact relationships. If the radii of curvature of the channel are altered from one point of inflection to the next so that they are smallest at the vertex of the curve and then

increase gradually (from the vertex to each point of inflection) to infinity again at the points of inflection, then the pools will be deepest in the vertex of the curve and will gradually decrease in depth as the inflection points are approached. In the same way, it is essential that the length of the curve be made to correspond to the inherent nature of the stream, by matching the deflection points with those of a well-developed reach of the natural river investigated.

The result of a well-guided curvature is generally to produce a satisfactory channel in which the talweg passes through the middle of the cross-sections at points of inflection and crosses in an easy curve to the outside of the next bend. The close relationship between radii of curvature and pool depths must be kept in mind whenever experiments with models are undertaken because, since the purpose of such experiments is to determine methods for procuring a good, navigable channel, the flow must be made to follow smoothly from one bend to the next and thus secure navigable depths at all points.

As it is scarcely possible in theory to predetermine how deep the pools in a natural stream will become as a result of contractions, or whether dredging may be required, so also is it difficult to predict with assurance what depths may be anticipated unless the entire investigation is subjected to model study. Where a depth scale is chosen, as, for example, 1:50, there is no reason to expect that the same scale will be exactly applicable to the bed formations. By trial and accident this may be true, but in the average case compensatory factors must be determined and applied if the results are to be susceptible of quantitative as well as qualitative interpretation. The method most generally followed is to shape the model bed flat and, running through typical hydrographs for sufficient time to produce equilibrium in the bed movement, make observations as to the development, and comparisons as to its similarity with the corresponding developments of Nature. In many cases the natural development of the model is so nearly like that of Nature, that maps of the two are almost indistinguishable.

When a model is constructed to very small scale (as is frequently necessary) investigators are confronted with the aforementioned difficulty that sand, sufficiently fine to simulate (in scale) materials of the natural stream, is neither obtainable nor satisfactory, due to resulting riffles. To avoid the formation of riffles, many experiments have been made in America and abroad to determine the practicability of using other materials, such as ground lignite, saw-dust from pressure-creosoted timber, crushed coal, and other likely substances. A more satisfactory procedure has been found, however, in the use of coarser sand and increased tractive forces through greater slopes. In doing this the velocity of flow, of course, must not be made to exceed the upper limit for turbulence, above which shooting flow will be produced. This type of flow will result in the formation of standing waves, which are not characteristic of the type of flow ordinarily encountered in rivers.

If the water depth in the model is represented by D_m, the sand-grain diameter by E_m, and the roughness factor by C_m, then according to Professor

Krey, the limiting values for slope are: $\frac{E_m}{8\,D_m} < S_m < \frac{G}{C^2_m}$. Assuming the average value of C to be 80, then $S < \frac{1}{200}$. This value of 0.005 for the slope is much too great, wherefore, with the small values for D_m and E_m, riffles must undoubtedly appear when S_m is taken approximately equal to $\frac{G}{C^2_m}$; but, if a coarse sand (with a small value of C_m) is used, the obstacle will be largely eliminated. It appears, and is, possible, therefore, to choose E_m and S_m so that, instead of riffles, true sand banks will develop. In this same connection Armin Schoklitsch[4] states that the model sand should be compounded of various grain sizes. Natural sand banks are characterized by a diversity of grain sizes and, therefore, are firmer, for the reason that the spaces between the coarse and fine sand grains are in part filled and cemented with fine sedimentary material. To insure streaming (non-shooting) flow, S_m must be less than $\frac{G}{C^2_m}$, as is discussed in a subsequent paragraph.

Experiments Involving Movable Beds.—Material particles on the bed surface of a stream are acted upon by a number of conflicting forces, including those developed by the flowing water and those of a re-active nature arising from mass resistance and friction. Turbulence of the water may result in the exertion of component forces in several directions and may even produce an actual lifting of any reasonably light particle. This is the phenomenon known as transportation by suspension and is worthy of considerable study in connection with problems of sedimentation and kindred effects. From the standpoint of the practical river engineer, however, the phenomenon meriting most serious consideration is that of the movement of material particles along the stream bottom by hydraulic traction because, if the tractive force is sufficient in any case to overcome the inertia of a particle and its frictional resistance, movement in a down-stream direction will occur. When all bed particles, including the largest and heaviest, have been put into motion, general movement is said to have taken place, and, corollary to this, the critical tractive force for any particular bed load is defined as that force which will produce general movement of the material. ("Geschiebe" is a concise and descriptive German word used to designate bed material. Literally, it means, "that which is shoved.")

The tractive force of flowing water is nothing more than the component in the direction of movement of the weight of the water. Application of the laws of mechanics shows that this tractive force, per unit of area on the river bottom, is expressed by:

$$F = J \times D \times S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which, F is the critical tractive force per unit area; J is the weight of

[4] "Geschiebebewegung in Flüssen und an Stauwerken," von Dr. Techn. Armin Schoklitsch Wien, 1926.

water per unit volume; S is the slope (non-dimensional); and D is the depth. Tractive force, then, is a function only of slope and depth. This is a very important consideration and should be kept in mind.

Suppose that it is desired to design a model involving a movable bed. The usual considerations of space, water supply, etc., have caused the adoption of definite horizontal and vertical scale ratios, assumed for the sake of argument, as 1:500 and 1:125. It is requisite that the bed load in the model move in a manner similar to that in its prototype; hence, if general bed movement in Nature begins at a 30-ft stage, it should begin at a corresponding stage in the model; but at what stage does bed movement begin in Nature? It is seldom practicable to determine this by actual observation, due to the depth and turbidity of the average river. Fortunately, a solution by laboratory methods is readily found by applying the law of M. P. du Boys[5] as given previously herein, since it may be demonstrated that for any particular bed material the critical tractive force is constant; that is, if the slope is doubled, the depth may be reduced by one-half and movement will still obtain.

To return to the problem of determining where general movement occurs in Nature: Suppose a representative sample of bed material is obtained from the actual river and moulded to a constant depth in a tilting flume. Obviously, if the flume is tilted to the average slope of the natural bed, to get movement it would be necessary for water to flow over it at a depth equal to that at which movement occurs in Nature. Since this depth could not be obtained in the average laboratory flume the critical tractive force must be determined by increasing the slope in the flume until general movement begins at less depth. Then, knowing the slope in that particular reach of the river, and by substituting this in Equation (13), it is possible to determine the depth in the river at which general movement begins. Computations and procedure for the hypothetical case are, as follows:

Obtaining a large sample of bed-load material from Nature at the locality in question, where it so happens that the average slope is 0.0001, it is placed level in the flume to a depth of about 4 or 5 in., after which the flume is tilted to a slope of 20×0.0001, or 0.002. Then water is passed over the bed material, keeping the water-surface slope equal to the bed slope and increasing the depth until movement occurs. Assume that general movement occurs at a depth of 0.5 ft. This indicates that movement in Nature will occur at a depth of about 20×0.5, or 10 ft. Therefore, movement in the model must occur at a model depth of $\frac{10}{125}$, or 0.08 ft.

Next, select the sand it is desired to use for the model "geschiebe." Suppose, for example, that it is desired to use Mississippi River sand from a reach of the river near Vicksburg. In the tilting flume, it is found that for general movement a tractive force of 0.008 lb per sq ft is required with this material. It has already been stated that movement is desired in the

[5] *Annales des Ponts et Chaussies*, Tome XVIII, 1879.

model at a depth of 0.08 ft; hence, in Equation (13): F = critical tractive force = 0.008 lb per sq ft; J = weight of water per cubic foot = 62.5 lb; D_m = depth = 0.08 ft; and, therefore, S_m = 0.0016 ft per ft. From this it follows that proper movement in the model will result if it is given a bottom slope of 0.0016, or sixteen times that of the natural river reach. Since there is a vertical distortion of 4 to 1, the slope of the model would normally be 4 × 0.0001, or 0.0004. Thus, it appears necessary to supply the additional slope by some approved method. The final slope distortion will be, $\frac{0.0016}{0.0001}$ = 16.

It is to be noted here that a tacit assumption was made, to the effect that the river bed is parallel to the water surface. That assumption is no doubt true for all practical purposes considering a long reach of a river. All it implies is that the mean depth of the river is fairly constant. However, there are pools and crossings in the river, and the bottom is not parallel to the water surface, except for average conditions over a long reach. Since one cannot hope for sufficient data to design for each crossing and each shoal, the best that can be done is to take averages, and, in the general case, such procedure should result in reasonable accuracy. Consider in this connection, that the model, too, has its pools and crossings and, hence, the average condition designed for will no doubt cause movement in the model at about the same time and place as in the prototype.

To insure proper movement of the model bed material the foregoing procedure may be epitomized, as follows:

1.—Select vertical and horizontal scales so as to produce as little distortion as may be consistent with the production of measurable depths and turbulent flow.

2.—Make a flume test of the sample of bed-load material obtained from the actual locality to be represented by the model, and determine the critical tractive force necessary to insure its transportation. Should it prove impracticable to obtain the bed-load sample in quantities sufficient for testing in the flume, or in the event equipment is not available, the critical tractive force may be obtained by the use of any one of several formulas. Probably the best of these formulas is that of Kramer,[6] namely, $F = \frac{100}{6}\left(\frac{(j_1 - j)}{B}\right)E$, in which, j_1 is the specific gravity of the bed material; j, the specific gravity of water; E, the mean grain diameter; and B, a sand uniformity factor obtained by plotting the curve of the mechanical analysis of the bed material, separating the area under the curve into two areas—above and below the 50% line—and determining their ratio.

3.—From a consideration of the results of Step 2 and of the average slope of the natural river, determine the depth at which bed movement occurs in Nature.

[6] "Modellgeschiebe und Schleppkraft," von Hans Kramer, *Mitteilungen* der Preussischen Versuchsanstalt für Wasserbau und Schiffbau, Berlin, Heft 9, 1932.

4.—Determine the critical tractive force required for the bed material which will be used in the model. The method to be adopted may be either of those described in Step 2.

5.—From Step 3, plus a consideration of the vertical scale, the depth in the model at which movement is to occur, is known. From a consideration of this model depth and of the results of Step 4 determine the slope necessary for the model.

6.—Tilt the model sufficiently to obtain the slope found in Step 5.

Turbulence in Open-Channel Models

Requirements for Hydraulic Similitude.—In hydrodynamics many problems arise that defy purely mathematical solutions; however, hydraulic models now provide ready means for investigating such problems. The principles of hydraulic similitude insure that the results obtained from investigation by means of the model will be transferable, quantitatively or qualitatively, to the prototype.

Geometric similitude is concerned with the relations between linear dimensions of model and prototype. Chief among the factors influencing the selection of the horizontal (length and width) scale for the model are the area and shape of the prototype, laboratory space available for the model, and cost of construction. Prominent among the factors influencing the selection of the vertical (depth) scale is the range in elevations of the prototype as against the depths necessary in the model. The latter limitation leads to a consideration of dynamic similitude.

Kinematic similitude requires that every particle in model and prototype shall describe geometrically similar paths in corresponding time intervals; and, closely akin to this, dynamic similitude requires that these motions shall be produced by forces acting in a definite model-to-prototype ratio. For all practical purposes, dynamic and kinematic similitude exists when the flow of water in the model is of the same character as the flow in Nature. If laminar or stream-line flow exists in Nature, it must be reproduced in the model, whereas, if turbulent flow occurs in Nature that kind of flow must be reproduced in the model. As a matter of fact, flow in natural channels is almost invariably turbulent. There are index numbers, derived experimentally, by means of which model and prototype may be examined to find if there is dynamic similitude with respect to any particular force.

It is important to note that both gravity and friction act on flowing water, whereas, in general, dynamic similarity can exist only in respect to a single force. Evidently, this gives rise to a conflict of requirements, and perfect similarity is not to be attained. The best that can be done is to select that force which is of predominant importance, and to seek dynamic similitude with respect to that force. The modifying effect of the other force will prevent the accomplishment of absolute similitude, which in practice is not necessary. Fortunately, the effects of one or the other of the forces is usually so predominant that little error results in considering it as acting

alone. Thus, in experiments in which, a high degree of turbulence results, internal friction may be neglected and the laws of gravitation considered as governing.

Reynolds' Criterion for Turbulent Flow.—The index by which turbulence may be measured, was discovered in 1883 by the great English experimenter whose name it bears. The experiments of Osborne Reynolds were confined to small, smooth pipes, but sufficient evidence was obtained to show that a rather definite transition occurs between laminar and turbulent flow. It was further proved that this transition is indicated by a definite value of the non-dimensional expression, $\frac{VD}{K}$ (since called Reynolds' number), in which, V equals velocity; D equals pipe diameter; and K, the coefficient of kinematic viscosity.

From the expression, $R = \frac{VD}{K}$:

$$R = \frac{VDp}{u} \qquad \text{.....(14)}$$

in which, the coefficient, K, is replaced by its equivalent value, $\frac{u}{p}$ (viscosity divided by density). Since the dimension of V (velocity) is length per unit time, or $L\,T^{-1}$; of D (diameter) is length, or L; of p (density) is weight per unit value, or $M\,L^{-3}$; and of u (viscosity) is $M\,L^{-1}\,T^{-1}$, Reynolds' number has dimensions; that is, $\frac{L\,T^{-1} \times L \times M\,L^{-3}}{M\,L^{-1}\,T^{-1}} = M^{o}\,L^{o} = 1$, and the modulus may be shown to be merely a number, or ratio.

Table 3 gives values for absolute viscosity and kinematic viscosity of water for temperatures within possible working ranges.

TABLE 3.—VISCOSITY OF WATER

Temperature, in degrees Centigrade	Absolute viscosity, u, in English units	Kinematic viscosity, $K = \frac{u}{p}$
5	0.001020	0.00001639
10	0.000880	0.00001411
15	0.000766	0.00001230*
20	0.000600	0.00001085
30	0.000538	0.00000865
40	0.000442	0.00000713
50	0.000370	0.00000599

* Value generally used in computations.

The experiments of Reynolds being confined to small glass pipes, his determinations for the value of the viscosity modulus are not strictly applicable to open channels, but it is worth noting that in the case of pipes the modulus is equal to about 2 500. To write the expression so that it may be used in open-channel determinations, it is necessary, of course, to change pipe diameters into hydraulic radii. It should be noted that as these are both linear dimensions no change is made in the dimensions of the modulus.

From $\mathsf{R} = \frac{VD}{K} = 2\,500$, it follows that $\mathsf{R} = \frac{4\,VR}{K} = 2\,500$; or, $M = \frac{VR}{K} = \frac{2\,500}{4} = 625$, in which, M is the viscosity modulus for open channels, corresponding to Reynolds' number for pipes.

The value for the modulus (625) as determined from the Reynolds' number, is the subject of some disagreement among modern investigators, Schoklitsch setting its value at 900, and Hopf at 320. Probably, the safest value for its higher working limit is that established by the Prussian Research Institute for Hydraulics, Marine Design and Earth Testing, which gives 6 000, with a notation that an ample factor of safety is included. Since the range of turbulence is wide between laminar and shooting flow, it is obviously better to choose a safe value if it can be done without too greatly increasing the cost of the model. Accepting tentatively the value of the Prussian Institute: $M = \frac{VR}{0.0000125} = 6\,000$; or, $V\,R = 0.075$, in which, R is expressed in feet, and V, in feet per second. From this it may be assumed that turbulent flow will always exist when the product of velocity and hydraulic radius (or mean depth) is greater than 0.075.

Recent experiments conducted at the U. S. Waterways Experiment Station have thrown additional light upon the conditions necessary for production of turbulent flow in a small stream or model. The tests were made with a tilting flume, lined with concrete of the same general roughness that is used in models built at the Station, and curves were determined to show the variations of velocity with depth when slope was maintained constant. A straight-line relationship was observed to the point where turbulent flow occurred, the curve beyond this point becoming representative of a second degree function. Thus, the break from straight to curved line indicated the transition from laminar to turbulent flow and served to supply a more accurate criterion for turbulence than had been available previously. By averaging the results of many tests at different slopes, the value of the product of depth and velocity was found to indicate definitely the character of flow produced and that where the value was greater than 0.02 (in English units) turbulence prevailed. Expressed mathematically the criterion for turbulent flow is:

$$Z = V\,D > 0.02 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

D being expressed in feet and V, in feet per second.

It is possible, of course, for flow in the model to pass upward through the turbulent stage and into shooting flow. This possibility must be investigated, and, since the velocity of shooting flow is the same as wave velocity, $\sqrt{D\,G}$, in which, G is the acceleration due to gravity, the necessary condition to insure non-shooting flow is that the velocity be less than $\sqrt{G\,D}$. To summarize, turbulent flow in the model exists when $\frac{0.020}{D} < V < \sqrt{G\,D}$.

The discussion of geometric similitude has developed the fact that the depths necessary in the model govern the selection of the vertical scale. The discussion of Reynolds' criterion reveals the connection between model depths (hydraulic radii) and turbulent flow. To insure a high degree of turbulence in models of large rivers, such as the Mississippi, it is almost invariably necessary to use a scale value of depths larger than that for horizontal dimensions. This results in distortion of the model.

To summarize briefly: The horizontal dimension scale of a model is arbitrarily selected, usually with a view to making the model fit in the space available for it. The vertical scale is more intimately connected with the requirements for dynamic similitude, and must usually be made larger than the horizontal scale. On the typical river model turbulence is to be tested by use of Reynolds' number, the viscosity modulus, M, or the turbulence criterion, Z.

To make application of the foregoing principles clearer the reader is referred to an actual example published elsewhere[7] by the U. S. Waterways Experiment Station, which demonstrates how turbulence was considered in connection with a model of the Ohio River.

Practical Laboratory Expedients

Necessity being the mother of invention it would seem far better to complete the work of equipping a laboratory over an extended period as demand indicated than to complete the equipment at the very beginning. Heavy, fixed equipment is expensive, difficult to manipulate, and, unless chosen with discretion by a well-informed and experienced individual, may be inapplicable to the special needs of the proposed institution. Of course, in the case of laboratories designed for instruction of University students, it is more necessary to install a certain amount of fixed equipment in the beginning, and, in general, past experience may be used as a guide to such purchases. The set-up of a practical river laboratory, on the other hand, must be so flexible as to permit it to deal with any problem falling within its general scope, and the first essential to this is adequate and unencumbered space.

Special flumes, designed to carry river models are relatively worthless for the reason that they are adaptable only to special problems. Where one model may be 12 ft wide and 100 ft long, the next required may be 40 ft wide by 60 ft long; and, obviously, the same flume would be inapplicable to both. Moreover, the reach of the river under investigation may bend back upon itself in such a way as to preclude the use of a uniformly tilting flume. For a reasonably straight reach of river the tilting flume would be desirable in dealing with models requiring a slope greater than that resulting from vertical distortion; but if the river winds materially such slope must be acquired by the placement of successive templates. Even so, it is sometimes difficult to apply slope corrections, the situation being practically impossible when the river meanders and when over-bank flow must be considered. If flood stages are to be reproduced no slope may be given the model except that

[7] "Model Study of Effects of Dikes on the River Bed at Walker's Bar, Ohio River," *Paper L*, U. S. Waterways Experiment Station, 1932.

normally resulting from distortions, but if less than bank-full flow is the important thing, or if the reach to be investigated is reasonably straight, practically any increase in slope may be given. Fortunately, those problems requiring experimentation with extensive models are generally concerned with hydraulics of flow instead of bed movements, and increased slopes are non-essential to their satisfactory solution. In any case a temporary wooden box of sufficient size will make a container for the model, which may be built of sand and concrete. Alternate types of containers are built of concrete or brick and mortar.

The simplest method of constructing a model is to cut templates from sheet iron to conform to river and over-bank sections; level the containing box; and place the templates to grade by means of a level. The templates may then be braced and filled to within an inch of their tops with moist sand, packed in place. A coating of concrete, placed level to the tops of the tem-

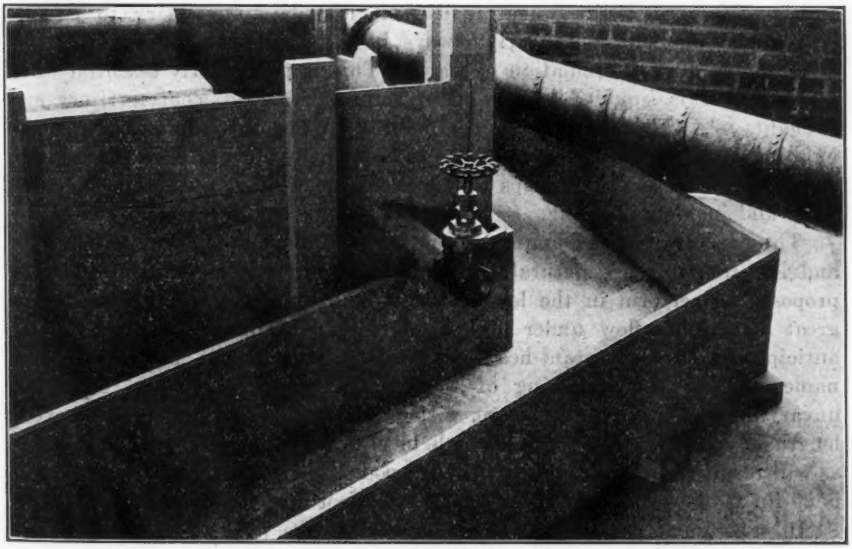

FIG. 1.—TAIL-GATE OF RACE-TRACK TOWHEAD MODEL, SHOWING VALVE OUTLET FOR FINE ADJUSTMENTS

plates and moulded to conform in shape with the sections, then completes the model proper, which must be equipped subsequently with an adjustable tail-gate, a measuring weir, an entrance chamber, and baffles. All these devices are extremely simple and except for the weir may be made by any good handy-man or carpenter. Practically any type of tail-gate is satisfactory to produce a back-water similar to that induced by resistance to flow of the entire river below the lowest point represented in the model. Fig. 1 is a model of a tail-gate, showing a valve outlet for fine adjustments.

Special flumes may be needed, of course, in a river laboratory for control tests and for tests of dams, weirs, spillways, and various measuring devices.

In this connection it is well to make the flume as long as the building will permit and of as great cross-section as is required to accommodate the maximum flow of water that can be allowed. Flumes of steel are generally more suitable than those of concrete because cracks and leakage may be developed in the latter as possible settlements in the building's foundations or as temperature changes occur. The former, moreover, may later be cut at any point should it be desired to insert plate-glass sections, or manometer connections. A smaller tilting flume may be built of wood or other material if suitably mounted on steel I-beams, or similarly rigid supports.

Having provided a laboratory building with expensive and unencumbered floor space upon which models may be constructed, and having designed an inlet and outlet system for the water, together with a diversion to some measurement chamber, little more need be taken into account until experiments actually have been begun. After work has started various needs will present themselves and ingenuity may be given free play in meeting requirements. If a reservoir system is used it may be unnecessary to provide pumping equipment, but nearly always it will be desirable to arrange for some pumping and circulation of water. This is particularly essential when the reservoir is supplied by run-off of rainfall from adjacent territory and droughts may occur during summer months. Whenever a pumping and circulating system is used it is necessary to provide also for: A deep (or storage) reservoir with overflow; pumps of required capacity; a constant head reservoir; and incidental piping.

The capacity of the pumps, size of the pipe, and dimensions of the high and low reservoir are naturally determined by the character of work it is proposed to perform in the laboratory, it being unnecessary to arrange for great volumes of flow under high head if only normal river problems are anticipated. The constant-head reservoir—having as its purpose (as its name implies) the supplying of water from pumps to models under an unvarying pressure and, therefore, without fluctuations in flow—must have a length of overflow crest great enough to eliminate, entirely, such variations as will result from pump pulsations. A good value to use as a basis of design is 20 lin ft of overflow crest for each second-foot of water pumped.

In subsequent paragraphs other accessories of a modern river laboratory are described. These descriptions are general and it should be kept in mind that any other workable design may be equally good in any case. The personnel should be encouraged to think and develop new ideas as new problems present themselves. Listed briefly, the accessories are:

(*a*).—*Level Pedestal.*—Since a level must be used continually for setting templates, checking elevation of model beds, and setting gauges for zero readings, a fixed pedestal mount is extremely convenient and helpful.

(*b*).—*Profilograph.*—This is a labor-saving device for obtaining cross-sections of the model river bed after running (see Fig. 2). It consists of a carriage that travels along a rack and holds a vertically sliding shaft equipped with a pencil at its upper end. The shaft is counterweighted and placed between nearly frictionless rollers so that as a roller at its lower end is

moved across the model bed the shaft, with its pencil, will rise and fall. Geared to the main carriage is a secondary carriage fastened to a vertical board on which a sheet of cross-section paper may be mounted. Movement of the main carriage results in reduced movement of the secondary carriage in relation to it, so that with the pencil moving vertically and the paper

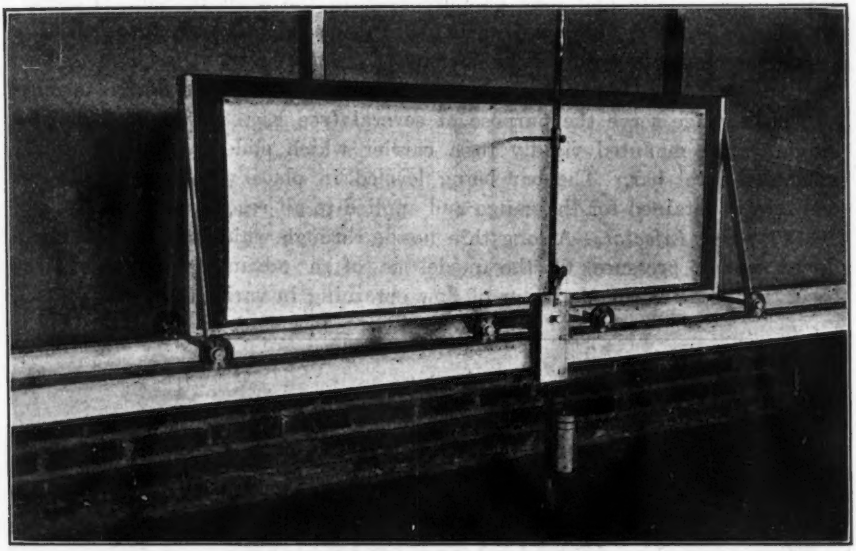

FIG. 2.—PROFILOGRAPH FOR TAKING CROSS-SECTIONS OF MODEL RIVER BEDS

horizontally a section of bed is traced to full vertical and reduced horizontal scale. A more direct and efficient method of obtaining the same result has been recently developed, by utilizing a special sounding rod equipped with a pivoted foot and an adjustable T-head that rides on a leveled cross-beam.

(*c*).—*"Geschiebe" Meter.*—This is a simple device designed to feed bed material to the model at a fixed rate and in accordance with the movement of such material in the model. Operating on the hour-glass principle with funnels of various sizes to be used interchangeably, it is free from moving parts and is very effective. The bed material is washed into the model by a small stream of water brought in by a rubber tube.

(*d*).—*Current-Measuring Devices.*—These devices are of many types and designs, the simplest expedient for velocity determinations being the use of floats of varying weights and densities. Other devices are pendulums actuated by the velocity of flow, small propellers mounted between jeweled bearings, and Pitot tubes.

(*e*).—*Depth Meter.*—Difficulties are frequently encountered in determining depths of flow in very shallow reaches of a model, where, as in a floodway problem, many such determinations must be made. By placing round-headed gutter nails in the model so that the heads project but slightly through the concrete a needle-gauge mounted on a U-frame may be used to

determine differences between elevations of the nail-heads and the water surface. Data are obtained for each nail-head and with the depth meter measuring the distance between the head of the nail and the water surface in each case, elevations referred to a common datum are readily determinable.

(*f*).—*Traveling Crane.*—Traveling cranes of many types and purposes are made commercially and may be purchased in the open market. Purchase should not be made, however, until a definite need has been revealed, after which the choice of type will be obvious.

(*g*).—*Multiple-Gauge Rack and Wells.*—This is a simple device to make one hook-gauge serve the purpose of several (see Fig. 3). It consists of a single gauge mounted rigidly to a carrier which slides along a carefully machined steel bar. The bar being leveled in place with leveling screws, one zero is obtained for the gauge and applied to all readings obtained.

(*h*).—*Dye Injector.*—A long thin nozzle through which dye may be introduced under pressure to the model is often advantageous in making observations as to the character of flow obtaining in various reaches.

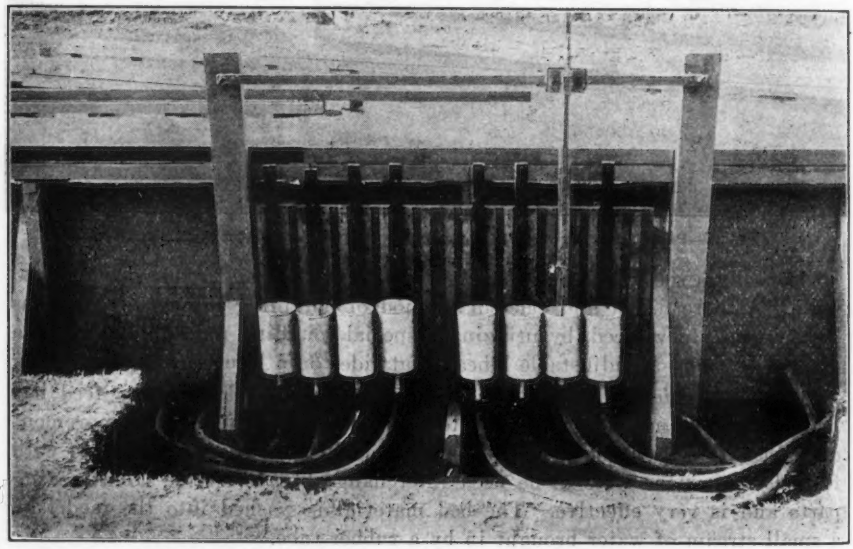

FIG. 3.—GAUGE RACK WITH MULTIPLE-WELL ARRANGEMENT

(*i*).—*Racks and Baffles.*—These may be of many types and their design in any case should be the result of trial, experience, and experiment. Wooden gratings, crushed rock, coarse gravel, baffle-plates, wire screening, and swinging boards may all be used as the case demands.

(*j*).—*Valve Outlet at Tail-Gate of Model.*—This arrangement, shown in Fig. 1, is often advantageous in controlling the flow from the model. After a rough setting of the gate, fine adjustments of flow are possible by regulating the valve.

Typical Experiments Encountered in a Rivers and Harbors Laboratory

Classification of Model Types

Fluvial models may be classified according to the character of their beds and according to the nature of their predominant forces, thus:

1.—Fixed Bed Models.—

(*A*) Those for which a consideration of gravitational force is paramount, and to which Froude's law must be applied, either: (*a*) Strictly; or (*b*), where channel friction is of decisive influence, by application of a factor to adjust the velocity and discharge scales.

(*B*) Those for which a consideration of internal friction is paramount and to which Reynolds' law must be applied.

2.—Movable Bed Models.—

Same as Item 1(*A*) and Item 1(*B*). For this type of model, consideration must also be given to a development of tractive force comparable to that existing in Nature according to the kind of sand used.

The preceding outline classifies in the briefest possible form the several types of models that may be used for different investigations. However, since every problem will require a different treatment it does not follow that a model must be built in one of several ways and that no other methods are permissible under any circumstances. It is intended to show, on the other hand, that many possible combinations of conditions must be considered and that while there are typical model forms, these must be varied sufficiently in each case to meet the peculiarities of the problem. Several general types of models suggested by the outline will now be discussed.

Type 1(A(a)).—Fixed Bed Models to Which Froude's Law Must Be Strictly Applied.—This type of model is exemplified by those used for determinations of: (1) Discharge capacities of weirs, spillways, siphons, etc.; (2) effects of sills on aprons of overflow structures; (3) factors influencing the production of an hydraulic jump; and, (4) other hydraulic phenomena resulting from the gravitational fall of water when friction may be largely neglected.

Type 1(A(b)).—Fixed Bed Models in Which Channel Friction Is of Decisive Influence.—Typical of experiments requiring this class of model are those designed for determinations of effects of cut-offs and limits of back-water. These problems, being practically independent of channel changes, may be solved satisfactorily with a fixed bed in which experimental runs may be repeated time after time for the assurance of accuracy. When it is desired to obtain effects of silting or erosion, critical parts of the model may be built in of sedimentary materials. The problem of back-water or its corollary is frequently offered to a laboratory for solution, and it will be worth while for the reader to study two brief reports of the U. S. Waterways Experiment Station on this subject.[8]

[8] "Experiment to Determine the Limit of Backwater Influence in the Illinois River," *Paper Y*, 1931, and "Experiment to Determine the Effects of Mississippi River Backwater on the Red River," *Paper 10*, 1933.

The problems described in these reports resolved themselves into simple determinations of water-surface profiles after adjustment of the models had been accomplished, and no question of bed movement was involved in the investigations. Cut-off problems and similar determinations involving the addition or subtraction of resistance to flow are pursued in an identical manner, the questions of erosion and sedimentary deposition being subordinated to questions of effects on stages within the reaches considered.

Type 2(A(a)).—Movable Bed Models in Which Gravity May be Considered the Controlling Force.—Experiments with models of this nature will generally follow the lines of those mentioned in connection with models of Type 1(*A*(*a*)); the main difference being that erosive effects must be taken into consideration. Investigations conducted by Antonin Smrček, in Brno, Czecho-Slovakia,[9] and by Theodor Rehbock, in Karlsruhe, Germany,[10] to determine relative scour below spillway aprons, involved typical models of this classification.

Type 2(A(b)).—Movable Bed Models with Channel Friction the Controlling Force.—Models built for the determination of design of contraction works of permeable type are illustrative of this classification. In the general case, where qualitative effects are sought in preference to less easily obtainable quantitative data this type of model will suffice adequately. Thus, if it is desired only to choose between several proposed plans of contraction works, a movable bed model, built with primary regard to consideration of the force of gravity, is indicated. In this case rates of bed movement in model and Nature are observed, and corresponding bed changes under the various imposed conditions are compared.

In practical experimentation there occur instances in which the foregoing classifications cannot be applied rigidly. Most common is the case of flow over a regulating work, where the question arises as to whether the channel friction is of appreciable influence. For a method by which compatibility between the laws may be attained in a model, the reader is referred to a discussion by Lieut. John Paul Dean, U. S. Corps of Engineers, and the writer.[11]

The types of study indicated under 1(*A*(*b*)) and 2(*A*(*b*)) of the preceding classification are those in which the forces of internal friction of the fluid are predominant. Typical of such studies is the problem of the resistance to motion of a deeply submerged body, such as a submarine. The laws governing this type of experimentation are not compatible with those governing the cases heretofore discussed unless fluid different from that of the prototype is used in the model. Fortunately, in the field of practical river hydraulics, the forces of internal friction of the liquid are relatively small and may be neglected without appreciable error resulting. It may be noted that the more turbulent the regimen of a stream becomes the more nearly negligible are the forces of internal friction.

[9] Wilsonova Přehrada Na Rece Tennessee, Alabama, U. S. A., by Prof. Ing. Antonin Smrcek, v Praze, 1929.

[10] "Hydraulic Laboratory Practice," by the late John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., 1929.

[11] "Geometric Versus Hydraulic Similitude," *Civil Engineering*, August, 1932, p. 467.

Problems Susceptible to Laboratory Treatment

Experience indicates that the following general types of problems may be prosecuted successfully in river hydraulics laboratories: (1) Effects of dikes or other contractive works on the river bed; (2) effects of cut-offs of river bends; (3) back-water influences; (4) effects of channel dredging; (5) relationship existing between mean discharge and river sections; (6) discharge over navigation dams and spillways; (7) design of locks and dams; (8) determinations of velocities and current directions in rivers after placement of corrective works; (9) flow through siphons, siphon spillways, etc.; (10) effect of baffle-piers on spillway aprons; (11) loss through trash-racks and screens; (12) effect of storage basins on flood flow; (13) progression of flood waves; (14) flow through diversion floodways; (15) effect of bends in a river; (16) harbor entrance problems; (17) resistance of, and flow around, cylinders and bridge piers in streams; (18) effects of flow over highways and railroad embankments; (19) experiments to determine effects of obstructions placed in a stream (submerged sills, etc.); (20) flow of water through sluices of various kinds; (21) towing experiments and experiments on bodies falling in water; (22) experiments relating to transportation of sedimentary and bed materials; (23) design of stilling-pools; (24) movement of sedimentary material and its distribution at river divisions; and (25) seepage in earth embankments and levees.

Types of investigations of the future are difficult to predict, but the definite tendency is toward more and more work with models of the movable bed type. In this field the surface has been scratched by experiments similar to that performed to obtain a solution of the difficulties encountered at Island No. 35, Mississippi River, but infinitely more remains to be accomplished. However, that may be, it still seems assured that much work will remain at all times for every hydraulic laboratory in the land.

Acknowledgments

It is desired to express appreciation to H. W. King, M. Am. Soc. C. E., and C. O. Wisler, Assoc. M. Am. Soc. C. E., for carefully reviewing this paper in an earlier form, and to acknowledge gratefully the loyal assistance and co-operation rendered by the Staff of the U. S. Waterways Experiment Station in developing new methods of laboratory technique described in the preceding pages.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EVAPORATION FROM WATER SURFACES
A SYMPOSIUM

Discussion

BY JOHN W. PRITCHETT, ASSOC. M. AM. SOC. C. E.

JOHN W. PRITCHETT,[37] Assoc. M. Am. Soc. C. E. (by letter)[37a].—While this Symposium brings together many evaporation data not heretofore collected in one publication, it is also noted that many evaporation records in Texas, covering long continuous periods, are not disclosed. This is doubtless true of evaporation data covering other sections of the United States. There are many evaporation records that have never been published, which would be of much value to the profession. The U. S. Weather Bureau would be a good agency to handle such work if allowed the necessary money.

Evaporation records beginning in 1915 or 1916 are being taken at eleven Texas Agricultural Experiment Stations. Records taken by the Bureau of Agricultural Engineering, in co-operation with the Texas Board of Water Engineers, at Mercedes, in the Lower Rio Grande Valley, are valuable in that comparison may be made of the evaporation from pans of different sizes and under varying climatological conditions for complete yearly periods. The land tanks were 1, 2, and 3 ft in diameter, and the floating tank was 2 ft in diameter.

Many of the Texas data are all-year records and on that account would be particularly valuable in carrying on evaporation studies. The San Antonio Field Station, Bureau of Plant Industry, U. S. Department of Agriculture, has an evaporation and temperature record beginning January, 1907. This is a complete 26-yr. record for a land pan (Type 2), 6 ft in diameter.

Table 5(*b*) shows a comparison of Pan (1) and Pan (4) at Austin, Tex. This is known as the Hill's Ranch Station, and is about five miles southeast of Austin. It was established by the Water Resources Branch of the U. S. Geological Survey in co-operation with the State Board of Water Engineers and with the U. S. Weather Bureau. The records were taken under the

NOTE.—This Symposium on Evaporation from Water Surfaces was published in February, 1933, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings* as follows: May, 1933, by Messrs. Ralph R. Randell, C. E. Grunsky, and Charles H. Lee; and August, 1933, by Messrs. J. T. Harding, L. Standish Hall, and Adolpho Santos, Jr.

[37] Office Engr., Texas Board of Water Engrs., Austin, Tex.

[37a] Received by the Secretary October 9, 1933.

direction of the District Engineers of the U. S. Geological Survey, and the computations were made by the Weather Bureau.[38]

This station was established on March 9, 1916. It is 475 ft above sea level, at Latitude 30° 14′ N, and is located about 1 000 ft from the ranch house. Observations are taken daily at 8:00 A. M., 90th Meridian time.

According to the U. S. Geological Survey,[39] the topography is rolling to the north and east and rough to the west and south, with a gradual slope to the Colorado River. In the immediate vicinity of the pan the slope is approximately 30 ft per mile. The reservoir is 30 by 200 ft, instead of 20 by 150 ft, as reported in the Appendix to Mr. Rohwer's paper, and the galvanized iron floating pan (36 in. square and 13 in. deep) is located 30 ft east of the land pan. It is freely exposed to the rays of the sun at all daylight hours.

This is probably one of the most complete records that may be found from which to determine the relationship between the evaporation from pans of Type (1) and Type (4). While the published records frequently show days missing or estimated records, the original records are practically complete. In cold weather when the pans were frozen over, the record was omitted in the publication and noted as days missing. The 14-yr. monthly record similar to Table 5, of Mr. Rohwer's paper, was tabulated by the writer, using all records, and using 32° F for the water surface temperatures when the records showed frozen water surface.

Comparing these results (not published) the writer has found that there is remarkably little difference between them and the data in Table 5. This would seem to indicate that a few years of evaporation record at a given station may suffice for all practical purposes.

The writer agrees with Mr. Rohwer and with the Sub-Committee that from the standpoint of making comparisons with known data on evaporation, the Class *A* Land Pan of the U. S. Weather Bureau (Pan (1)), is probably best adapted to the study of evaporation from water surfaces, because a large number of these pans have been installed in the United States, and many observational data obtained under similar conditions with a uniform procedure, are available from them.

In Table 10, Mr. Follansbee attempts to show the effects of factors influencing evaporation. In Table 10(*a*), showing the effect of temperature, the Washington, D. C., data are compared with the Austin, Tex., data. The 14-yr. record of the Austin Station shows a mean temperature of 67° F compared to 68° F, as shown in the table. The wind velocity at the pan, in miles per hour, is 2.1 compared to 2.0, as shown in the table. The relative humidity is 82 compared to 68. The evaporation for the 14-year record is 45.6 for both Pan (1) and Pan (4), applying a factor of 0.69 to the Pan (1) records and a factor of 0.90 to those of Pan (4). On account of the difference in relative humidity between this station and the Washington Station, the difference in temperature could not be said to acount for all the difference in evaporation.

[38] "Climatological Data, Texas Section," U. S. Weather Bureau, December, 1916.
[39] *Water Supply Paper No. 438*, U. S. Geological Survey, p. 31.

In comparing stations of approximately the same mean annual temperature, in order to show the effect of wind velocity and of relative humidity on evaporation, it appears to the writer that it would be necessary to know that the stations had not only the same mean annual temperature, but also the same range of temperature.

In establishing the coefficients to be applied to the different types of evaporation pans, in order to reduce the evaporation to the open-water equivalent, it is suggested that a more accurate determination might be made by finding coefficients that would apply to certain temperatures or ranges of temperature. Determining coefficients for individual months would have somewhat the same effect.

Various formulas have been derived from which evaporation estimates may be made that are sufficiently accurate for many practical purposes. The application of such formulas may soon be found to be a more practical manner of estimating evaporation, rather than basing such formulas on evaporation-pan records. Evaporation observations would then only be necessary for scientific purposes, or where extreme accuracy could be attained in obtaining climatological data.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

HIGH DAMS ON PERVIOUS GLACIAL DRIFT

Discussion

BY MESSRS. WILLIAM P. CREAGER, ADOLPH J. ACKERMAN, J. ALBERT HOLMES, AND IRVING B. CROSBY

WILLIAM P. CREAGER,[17] M. AM. SOC. C. E. (by letter).[17a]—A partial comparison of the Hardy Dam with the Soft Maple Dam is included in this excellent paper. It is of interest to make further comparisons since the two dams are similar in many respects. They have practically the same height and thickness, but the Hardy Dam is 2.7 times as long as the Soft Maple Dam. They are built of practically the same materials and, with the exception of the mudstone layers, the foundations are almost the same.

The chief source of difference lies in the use of a core-wall and up-stream blanket for the Hardy Dam and the mudstone layers in its foundation. The author gives the following data for seepage at the Hardy Dam:

	Initial	After six months
From the pond	3.7	. .
From ground-water	3.0	. .
Total	6.7	5.7

The seepage at the Soft Maple Dam is:

	Initial	After 1 year	After 8 years
From the toe of the dam	8	6	4.5
From the adjacent hill	6	4	3.0
Total	14	10	7.5

It will be noted that the seepage from the Soft Maple Dam is about twice that from the Hardy Dam despite its much shorter length. How much of this difference is due to the core-wall, to the up-stream apron, or to the mudstone layers in the foundation, is problematical. It will be noted that a large

NOTE.—The paper by Edward M. Burd, M. Am. Soc. C. E., was published in April, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: May, 1933, by Messrs. Charles W. Sherman, and F. B. Marsh; September, 1933, by M. M. O'Shaughnessy and H. de B. Parsons; and October, 1933, by Messrs. Joel D. Justin, and A. K. Pollock.

[17] Chf. Engr., The Power Corporation of New York, Buffalo, N. Y.

[17a] Received by the Secretary, September 28, 1933.

part of the seepage from the Soft Maple Dam came from springs in the adjacent hills, which seepage would be little, if any, affected by changes in the design of the dam. There is also evidence at the Soft Maple Dam that an appreciable quantity of the water seeping from the toe passes around the dam, the piezometers at equal distance from the top of the dam showing higher gradients near the ends than in the middle. In all probability not more than 25% of the total seepage is passing through the dam.

The effect of a core-wall in a dam on an equally or more pervious foundation is problematical. The writer made some experiments to determine the effectiveness of a cut-off under an impervious dam on a deep pervious foundation overlying rock. It was found that, for a cut-off extending 40% to rock, the seepage was reduced only 23% and even with a cut-off extending 82% to rock, the seepage was reduced only 50 per cent.

Therefore, a cut-off that extends only a relative short distance into a deep homogeneous pervious foundation has little additional effect on seepage compared with a core-wall that extends only to the surface. It increases the resistance to flow under and around the dam and, therefore, the slope of the hydraulic gradient passing the core-wall by virtue only of a slight reduction of area and a slight increase in percolation distance.

A core-wall at Soft Maple Dam, extending a practical distance into the foundation, would have reduced the total seepage certainly not more than 10 or 15 per cent. A blanket of fine material on the up-stream face and extending a considerable distance up the bottom and sides of the valley, would have been much more effective. The presence of mudstone layers at the Hardy Dam, if sufficiently extensive, into which the core-wall penetrates, may have had a considerable bearing on the lesser seepage.

ADOLPH J. ACKERMAN,[18] ASSOC. M. AM. SOC. C. E. (by letter)[18a].—The excellent paper by Mr. Burd is one of the first comprehensive records of an important phase of engineering which has been in the process of development during the past thirty years. The contributions by Michigan engineers to the field of dam design have remained comparatively unknown, and many of their well-established and sound principles, described in this paper, will prove surprisingly new and bold to others.

The author's use of the term, "sand dam," deserves particular emphasis in distinguishing it from the more frequently and loosely used term, "earth dam." Its distinctive characteristics are evident from inspection of Fig. 5 which shows the isolated position of the mechanical analysis curves with respect to those of more typical earth dams.

Fig. 7 is offered as a convenient basis for classifying the various types of natural-fill dams. It takes into account the full range of natural materials. Since the type of construction procedure, rolled, hydraulic, or semi-hydraulic (terms which are often used to designate the type of dam), is dependent chiefly on local and economic factors and, to a lesser extent, on the type of material, it is suggested that such designations, when used in combination

[18] Mankato, Minn.

[18a] Received by the Secretary September 25, 1933.

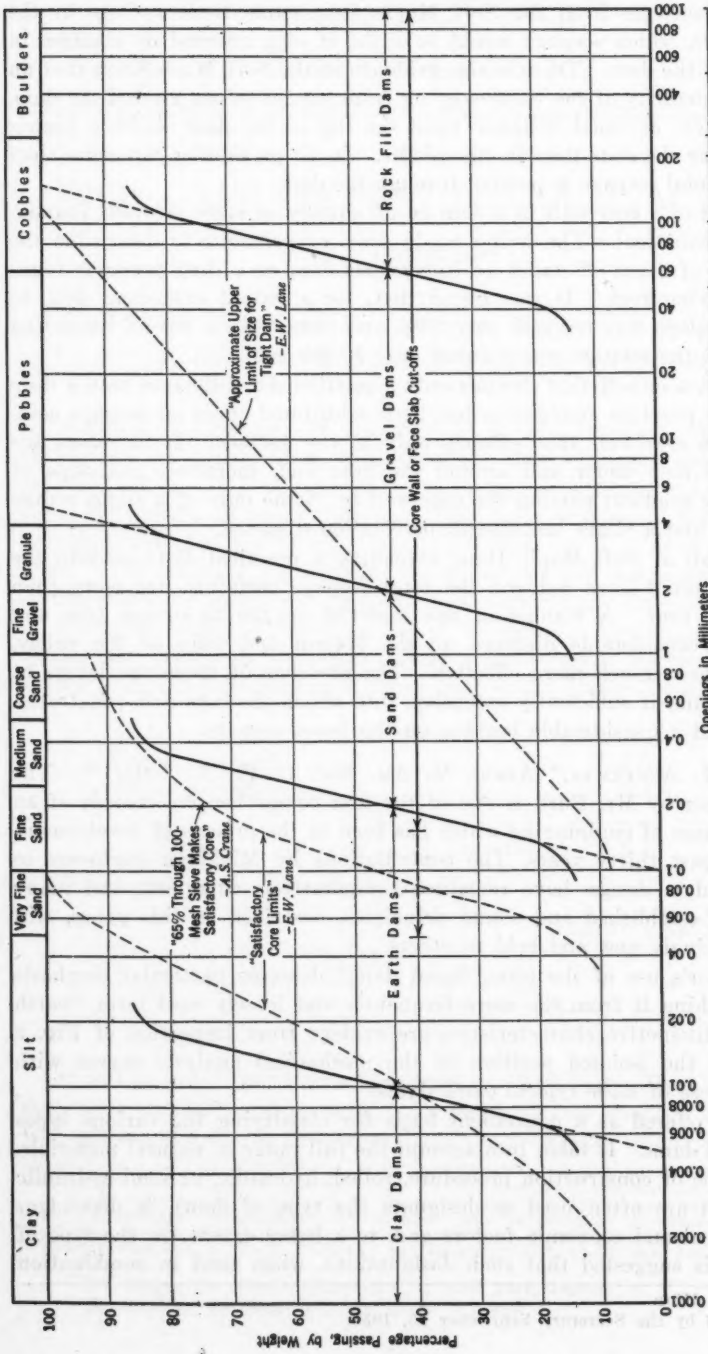

FIG. 7.—CLASSIFICATION CHART, NATURAL FILL DAMS

with the terms on the curve, will be more descriptive and positive in definition. Where several materials are involved, the practice of some engineers to use a combination name, such as "earth-gravel-rock-fill dam," is obviously more satisfactory than merely "rolled and dump-fill dam."

While a classification chart based on mechanical analyses has proved satisfactory for comparing materials of various dams, it must be repeated that the suitability of a material for a proposed dam is not defined thereby. The factors of solubility, "sloughing when saturated," ease of compacting, "sticking to roller," failure to consolidate rapidly, permeability, pumping limits, etc., depend upon other tests.

A number of controversies have been staged over the utility of core-walls in earth dams, and there are, unquestionably, several dams in which an abundance of impervious core material has made the adoption of a core-wall appear unduly conservative. The author has not only demonstrated the merits of a core-wall for the special conditions of his region, but has properly concluded that in a sand dam percolation characteristics are such as to require the same treatment of design that is applied to gravel dams, such as the Mackay Dam, at Pendleton, Ore., or, to a limited degree, to rock-fill dams; his recommendation, therefore, to place the cut-off diaphragm on the face of the sand dam is obviously logical and is certain to result in a more economical structure, as long as adequate safeguards have been taken in the design of the face slab to maintain its water-tightness.

The writer was employed from 1923 to 1925 on two of the projects under discussion, and recognizes the very practical features in the proposed improved sand embankment (Fig. 6). The proposal to sluice the material up stream on a slope of 1 on 8 is amply conservative, since measurements taken under similar conditions, where the material was allowed to run parallel to the axis of the dam, gave a slope of 1 on 20.

Where the intake and power-house are combined in a single box-like structure and serve as a section of the dam, the requirement of approximately uniform base pressure under full head, instead of the customary middle-third theory of stability, places special demands on the designer's ingenuity. As a rule, the base slab and intake walls are tied together with reinforcement to utilize the weight of the superimposed water, and parts of the structure not required for water passages are designed as cells and filled with sand.

On the project built in an area of quicksand, the writer was confronted with the perplexing fact that with the addition of a new lift of concrete the beginning of settlement invariably lagged several days behind the pouring dates. A satisfactory explanation has since been offered[19] by Professor Terzaghi who showed that in saturated foundations this time lag represents the mechanical action of squeezing out the water.

The intake tower described in the paper as a monolithic structure is of more or less standard type, but the foundation problems assume special importance when ice thrust is considered, and, no doubt, some additional details on this point would be of general interest.

[19] "The Science of Foundations—Its Present and Future," by Charles Terzaghi, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 278.

FIG. 7.—CLASSIFICATION CHART, NATURAL [illegible]

J. ALBERT HOLMES,[20] M. AM. SOC. C. E. (by letter)[20a].—About thirty years ago the members of a Board of Engineers averred they could find no precedent for building an earth dam, with core-wall, to a height of 150 ft, but that to a height of 70 ft an earth dam on a firm foundation, properly built of selected material, was as safe and lasting as a masonry dam.

The Board was mildly taken to task for following precedent so closely, and for forgetting that the manifest destiny of the engineer is to control and direct the forces of Nature for the benefit of Mankind. In this case, Mankind would have been greatly benefited, in his pocket, had the Board for the moment forgotten precedent and forced Nature to disclose a method of building a safe earth dam to a height greater than 70 ft. Precedent need not have been abandoned, however, because wider investigation would have disclosed the fact that several earth dams were in existence exceeding 100 ft in height and many more than 70 ft high. Two of the high ones, 120 and 125 ft, both without core-walls, were in California. Strangely enough, no technical description of any of these dams could have been found at the time. Engineering literature had "slipped," perhaps had not recognized the significance of dams of earth, although that was the material most anciently and continuously used in building dams and dikes. Quickly there came a flood of information, filling engineering periodicals and overflowing into books. By doubting, the Board of Engineers had served the art of earth-dam building a good turn; but dams of sand were not within its purview.

Clay is naturally the most impermeable of earthy materials, but it is not found everywhere in the world, and Michigan is one of the regions where it is not always available, from which to "select the best." Sand is available, however, and has been used there and elsewhere as a dam-building material. Its successful use is a demonstration of the "control and direction of the forces of Nature."

British engineers in India, obliged to make use of sand, the material at hand, have determined in advance the slope that percolating water would take in passing through that particular sand, and then have given the down-stream face the same or a flatter slope, helped out at the toe by an inverted filter, ending in heavy rip-rap.

The designers and builders of Hardy Dam, by experience and research, were thoroughly familiar with the characteristics of the construction material they must use. Their problem was the economic control of seeping water. The solution was to construct a flexible cut-off wall, supplemented by deep sheeting below, assisted by a rock-filled trestle and pipe drains at the down-stream toe.

Granular materials, like sands, are relatively incompressible when confined. If the grains can shift their positions, considerable change in volume can take place, as when a loose sand is compacted by ramming or shaking; but without the possibility of such a change of position, the volume of the mass changes very little under pressure. A few pounds of air pressure applied to a mass of sand in a rubber container will render the material as firm as a rock, thus demonstrating the ability of undisturbed sands to sustain heavy loads.

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[20] Cons. Engr., Chicago, Ill.

[20a] Received by the Secretary September 29, 1933.

The foundation materials under the valley floor were restrained from lateral motion, and it is not surprising that the settlements at Hardy Dam were so slight. Heavy loading of an area would cause horizontal bulging in the direction of less heavily loaded areas, but this was foreseen by the designers and care was taken that there should be no abrupt change in pressure, that is, changes that would cause shear and the opening of waterways in the foundation material.

Among the mechanical analysis curves in Fig. 5, those for the Hardy Dam and for the Bridgewater borrow-pit materials lie close together, especially in their lower sections, in the smaller sizes. Mr. Burd thinks that this might indicate that sluicing at Hardy Dam would have produced a core as impervious as those in the Bridgewater Dams. This is an illustration of the varying characteristics of materials that must be used in the construction of dams in different parts of North America. The author has described the Michigan dune sands of that glaciated region. Bridgewater, N. C., lies south of and beyond the advance of the ice sheets. The writer has described the geology of this region and the borrow-pit material in some detail in a previous paper.[21]

In the pit face this material still retains the markings and appearance of the original ledge, but can be excavated readily by steam shovel without blasting; it breaks up further in handling. The cores of the three dams sluiced from this material have effective sizes of 0.026, 0.028, and 0.030 mm. The grains of the Bridgewater materials are not hard like those of the glacial sands of Michigan, and under pressure in place in the dams, and under the pressure applied in connection with the seepage tests, may have broken down in part and produced smaller sizes and a more dense material in cores and test tank than the aforementioned sizes, determined from unpressed samples, would imply.

A sample of Michigan dune sand has given an effective size of 0.218 mm. The effective size of sand grains in Soft Maple Dam was about 0.17 mm.; but what is more indicative of porosity of a material is its uniformity ratio. The uniformity ratio of the dune sand was 1.65, indicating high porosity.

As an alternative method of construction Mr. Burd suggests the use of a single set of dumping trestles in the down-stream embankment and sluicing to the up-stream face. Sand and gravel, containing considerable fine material, can be controlled successfully on steep slopes by means of shear-boards or their equivalents. During the period, 1895 to 1897, the Northern Pacific Railroad Company placed, by sluicing, more than 600 000 cu yd of fill in fifteen trestles in the State of Washington, at an average cost of 6.39 cents per cu yd. The purpose of making the fills was to eliminate the trestles. The slopes approached more nearly those of railroad fills than the slopes of dams. The material was conveyed from pits to fills by ground-sluicing and sluice-boxes; no material was dumped from trestles.[22]

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[21] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 332.

[22] "Reservoirs for Irrigation, Water Power, and Domestic Water Supply," by the late James Dix Schuyler, M. Am. Soc. C. E., Second Edition, 1909.

While the sand and gravel could be held on the slope, the author suggests, it might not sluice on grades as flat as 1 on 8 without follow-up methods; that is, without using water under pressure at considerable distances from the trestles. At Wanaque Dam, in New Jersey, the sand and gravel, dumped from belt conveyors on the cut-off wall, was sluiced to place, but did not assume very flat slopes, except the smaller sizes.

The importance of consolidating sand and gravel fills with water cannot be too strongly emphasized. No other method will arrange the grains in large masses in so compact a manner as water. Even standing water, as by irrigation, will bring about remarkable compacting. Caleb M. Saville, M. Am. Soc. C. E., has demonstrated[22] what may occur, in the two methods of sluicing, to the up-stream face, or to the center of a dam. The line of saturation is always above or coincident with the line of complete saturation; that is, the line shown by standing water in bore-pipes or model gauges, unlike the hydraulic grade, begins at the junction of the dam face and pond-water surface. At times, especially at the toe, it is higher above complete saturation than the capillarity of the material. The line of complete saturation and the line of capillary saturation are both of interest and importance, but if and when used (and for clarity), they should be designated fully and correctly and not confused with or called the hydraulic grade.

From tail-water at the rock-filled trestle to pond-water surface at the splash wall (Fig. 1), the slope is about 1 on 2, or approximately parallel with the upper part of the embankment slope. A saturation line is shown on the cross-section of the dam (Fig. 1), apparently drawn from the top of the rock-filled trestle to the cut-off wall on a slope of 1 on 5. This slope is an average, determined by observation and the experience gained by the designers from other structures of similar materials built by them, and lies safely within the lines of the down-stream embankment. Its position depends on the integrity of the cut-off wall and the ample drainage afforded by the rock-filled trestle, and they, in turn, make possible the steep slopes and economic section of the down-stream embankment.

The writer's understanding of the "saturation line" and its position, as shown in Fig. 1, and on the proposed dam section having a paved up-stream slope (Fig. 6), is that with the drainage provided at the toe, saturation might take place to that line, at the ends of the dam, from hillside seepage and elsewhere, in case of leakage through the cut-off or paving. This type of flexible cut-off was developed previous to 1908 by Mr. Fargo, who is mentioned in the paper.

In the absence of a mudstone layer with which to connect cut-off sheeting at the up-stream toe (Fig. 6) an embarrassing quantity of seeping water might be forced up and into the coarser materials in the down-stream part of the embankment. Modern laboratory investigation of the influence that cut-off walls and their positions have on flow under sand dams, would furnish valuable information.

[22] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 97, Figs. 39 and 40.

The author's curve of total settlement of the crest wall (Fig. 4), shows, by a series of four dots just below the 3-in. line, a period of rest at the end of the embankment building period and the beginning of pond filling. This is a characteristic record of embankment settlement and the writer would expostulate with the author for not permitting his curve to follow the dotted line.

Foundation material does not settle appreciably until a considerable load has been applied. It is then rapid during loading and less rapid thereafter. The greatest percentage of settlement takes place at the base of the fill. To measure the settlement taking place during construction and at various depths, a series of plates and attached pipes were placed in the rolled fill embankment of Fifteen-Mile Falls Dam, on the Connecticut River, at Barnet, Vt.

The embankment is a clay and gravel structure, Fig. 8, having a maximum height of 162 ft and abutting against a high retaining wall at the foot of the east bluff. The space between the back of the wall and the foot of the

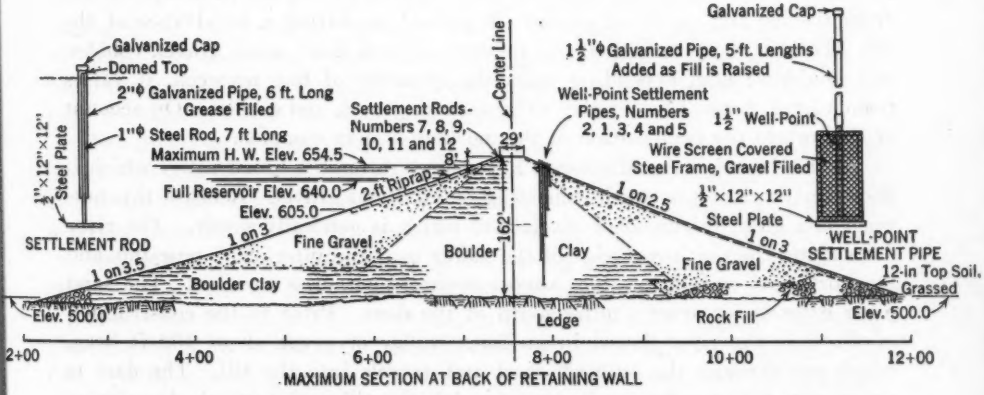

FIG. 8.—ROD AND WELL-POINT SETTLEMENT PLATES AND PIPES, FIFTEEN-MILE FALLS, LOWER DEVELOPMENT

bluff was narrow. Here, the boulder clay fill was placed by hand and air-tamped. The remainder of the fill, to the top of and on the bluff, was delivered in trucks, spread by "bull-dozers" and rolled in layers not less than 6 in. thick after rolling. The first plate and pipe, No. 1, was placed on the layer of air-tamped clay, 18 ft thick, at that point. Other plates and pipes were placed at 30-ft intervals in the clay fill, and immediately after the completion of the fill six plates with attached rods were placed in the top of the embankment. Well-points were attached to the deep plates and surrounded by a gravel-filled steel frame. This was for determining water elevations in the fill. Elevations of plates were obtained as each 5-ft section of pipe was added. Frost penetrates deeply in the latitude of Fifteen-Mile Falls and to keep below the frost line 7-ft rods were used with the plates placed in the finished embankment. For the protection of the rod and to prevent gripping and lifting by the frozen ground, a pipe is placed with the rod.

The grease in the pipe excludes surface water, which in freezing would disturb the rod and plate.

Experience at Fifteen-Mile Falls Dam demonstrated that total settlement in a well rolled embankment is not great. It occurs mainly during construction and is not recorded by elevations taken at the top of a dam after completion.

IRVING B. CROSBY[24], AFFILIATE, AM. SOC. C. E. (by letter).[24a]—Hardy Dam is on the Muskegon River where it passes between the great interlobate moraine, which extends up the western side of the Southern Peninsula of Michigan between the Muskegon and Manistee Rivers, and the combined Charlotte and Valparaiso moraines, which extend south from the Muskegon River, the first swinging to the east through Southern Michigan and the second swinging to the southwest around the southern end of Lake Michigan.

During the retreat of the ice sheet the interlobate moraine was formed between a lobe of ice which occupied the Lake Michigan Basin and a lobe to the east. The moraine consists partly of more or less sorted débris washed from the ice and partly of glacial till pushed up during a re-advance of the ice. The till is an heterogeneous mixture of rock-flour, sand, gravel, cobbles, and boulders; and, depending upon the quantity of fine material, it varies from a hard, dense, boulder clay to loose sand, gravel, and cobbles. On account of this origin the composition of the moraine is very complex.

At the dam site, the Muskegon River flows through a broad valley which is floored with glacial outwash consisting of sand and gravel. Beneath this outwash is a great thickness of glacial till which is generally sandy. The river flows along the northwest edge of the valley near the interlobate moraine, and the northwest end of the dam almost connects with the moraine. The sand plain extends for several miles south of the dam. Prior to the construction of the dam the river flowed in an inner valley or gorge about 100 ft deep, which cut through the outwash sand and gravels into the till. The dam is across this inner valley and connects with the till and outwash deposits at either end. Embankment material was generally taken from the outwash deposits, which are less heterogeneous than the till.

Hardy Dam rests directly on a layer of so-called "mudstone" which has a maximum thickness of 20 ft, but which apparently was thicker before it was eroded by the river. This "mudstone" is a very fine-grained sediment containing small shells, and, in its natural condition, is moderately hard and very dense and impervious. It dries out slowly and can be easily crumbled in the fingers in either a dry or a damp condition. This "mudstone" contains practically no clay and is not plastic. When dried and thoroughly crumbled, all but a small percentage will pass through a 200-mesh sieve. The coarser fraction consists of shells and fragments of shells and of fine sand, largely rounded grains of quartz. That part which passes through the 200-mesh sieve varies greatly in grain size, and the grains are mostly angular. It consists largely

[24] Cons. Geologist, Boston, Mass.

[24a] Received by the Secretary October 9, 1933.

of minute fragments of calcite with a small percentage of quartz and feldspar. This "mudstone" resembles, in composition, marls that are found in adjacent parts of Michigan, and it appears probable that this material is a marl formed by the chemical or organic precipitation of calcium carbonate in swamps. The fact that peat was found associated with it supports this view. The quartz and feldspar presumably were washed into the swamp and mixed with the calcite. The marl is overlaid by glacial till, showing that after its formation in a swamp it was over-ridden by an advance of the ice and subjected to great pressure, which accounts for its hard, dense condition. The till immediately overlying the marl is hard, dense, and reddish, and is quite different from the loose sandy till of the moraines.

The history of these deposits is apparently as follows: During the melting back of the ice sheet which covered Michigan in the last glacial period, great quantities of sand were deposited, and then swamps and ponds developed, in which marl and peat were formed. These deposits indicate the presence of animal and vegetable life. Then the ice re-advanced over the marl and peat swamps and laid down a layer of boulder clay which was compacted under the weight of the ice, forming the hard red till. During the subsequent melting of the ice the sandy till of the interlobate moraine was formed, and, finally, sand and gravel were deposited in a temporary lake at a much higher level than the lake in which the marl was formed.

Mr. Burd has emphasized the heterogeneity and irregularity of these glacial deposits and states that any but the most general deductions beyond actual exploration are wholly unreliable. It is true that with these glacial deposits little detail can be forecast without sub-surface explorations, but the results of such explorations can be interpreted more accurately by one who is thoroughly familiar with the geology of this type of deposit.

One of the most important problems in connection with a dam on pervious glacial drift is to estimate the seepage that will occur under and around the dam. With such irregular deposits this is difficult, but reasonably satisfactory results can be obtained. Estimating the seepage, however, requires thorough and accurate sub-surface information, undisturbed and unwashed samples, careful tests, and, above all, a thorough understanding of this type of deposits. Accurate boring samples are important, and wash samples are worse than useless. It is absolutely essential to obtain a sample from which nothing has been removed or to which nothing has been added by washing. If possible, an undisturbed sample with the original structure and compactness should be secured. With the best of boring samples, however, estimating the seepage under and around the dam is a much more difficult problem than estimating seepage through the dam, since in the first case the arrangement and condition of materials are at the best only imperfectly known, while in the latter case the arrangement and condition of the materials are controlled and can be known accurately. Since even elaborate sub-surface exploration does not give full or accurate information the best results can be obtained by a geologist experienced in this kind of work and who is familiar with the arrangement of this type of deposit.

Conditions at the Hardy Dam contrast strongly with those generally found at dam sites in the eastern part of Canada and the glaciated region of the Northeastern United States. At the Hardy Dam, and in a large part of the Southern Peninsula of Michigan, bed-rock is buried deeply, and the glacial deposits are thick and are generally of a sandy character. In this region, the present drainage systems have absolutely no relation to the pre-glacial drainage systems and are, with few exceptions, entirely independent of bed-rock control. At the Hardy Dam, and at other dams on the Muskegon River, bed-rock is at such great depth that it could not in any way affect the design of a dam.

In the East, the bed-rock topography is irregular as a rule, and the thickness of the glacial deposits varies greatly within short distances. In the more mountainous regions the streams are usually in their pre-glacial valleys, but not in their old channels, which are often buried at considerable depth. In the regions of low relief, however, the streams often pass from one pre-glacial valley to another and have little relation to the pre-glacial drainage systems; but even in these cases there are usually exposures of bed-rock, and its location often plays an important part in planning a dam.

The geological problems are not by any means the same in all parts of the Southern Peninsula of Michigan, and the conditions at Hardy Dam are probably as complicated as any, since it is at the southern end of a great interlobate moraine where till and sorted materials were both deposited.

In adjacent areas there may be dam sites with much simpler conditions. A site on an extensive outwash plain may have continuous layers of sand, gravel, or clay in a more or less orderly arrangement, and a site directly in a moraine may have foundations of boulder clay which would be tight and would offer no serious problems. The type of dam site and the nature of the problems can easily be determined by a brief geological investigation, and the details can then be investigated by borings and, if necessary, by permeability tests on samples. From all this information the geologist can then forecast conditions and answer the practical questions that will arise.

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DISCUSSIONS

ACTUAL DEFLECTIONS AND TEMPERATURES IN A TRIAL LOAD ARCH DAM

Discussion

BY MESSRS. LARS R. JORGENSEN, AND H. B. MUCKLESTON

LARS R. JORGENSEN,[11] M. AM. SOC. C. E. (by letter)[11a]—The Ariel Dam is a fine addition to the arch dam family and is well described in this paper. The work it represents is fully appreciated.

The cantilevers of the Ariel Dam have been calculated to take a certain part of the total water load, thereby relieving the arch for just that part. Accordingly, a dam with a slimmer cross-section has been obtained, than if the cantilevers had been left out of consideration for load-carrying purposes.

The general shape of the Ariel Dam, according to Fig. 4, remains as symmetrical as if the cantilever action had been neglected. In both cases, the up-stream and down-stream faces would be warped surfaces, bound by concentric circles at varying horizontal elevations, having lengths of up-stream radii decreasing toward lower elevations, similar to any ordinary constant-angle arch dam. Therefore, to all intents and purposes, the only difference between the actual design of Ariel Dam and one neglecting the cantilever action is, that the last-named would have greater thickness. With central angles the same in the two cases, the heavier section would have the smaller stress; thus, the problem finally simmers down to what the maximum stresses or the factor of safety ought to be.

Where a State commission or the Federal Power Commission dictates the maximum stress to be used in the design, as is the general rule almost everywhere, concrete can be saved perhaps by attempting to use the combined arch and cantilever method, but the same so-called saving, or the same thinner section, could have been obtained for the Ariel Dam by calculating the arch for the full load and increasing the arch stresses correspondingly. A small addition to the section near the abutments at the middle height of the dam would be of advantage to the design in any case.

NOTE.—This paper by A. T. Larned and W. S. Merrill, Members, Am. Soc. C. E., was published in May, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1933, by George Jacob Davis, Jr., M. Am. Soc. C. E.; and October, 1933, by Messrs. D. C. Henny, and B. E. Torpen.

[11] Cons. Hydro-Elec. Engr., Berkeley, Calif.

[11a] Received by the Secretary September 5, 1933.

The writer has visited the Ariel Dam and has never seen better workmanship. As far as the margin of safety is concerned it makes little difference whether the vertical cantilevers or the beams carry any load whatever. The arch stresses are kept low, according to data given in Tables 1 and 2, and the arch can safely carry much more load.

Only at the north abutment do the arch stresses given in Table 1 approach the allowable limit; but since the north abutment is only a continuation of the arch, except that the section of the continuation is about twice as large as that of the arch, the arch stresses determined under the assumption that the arch ends definitely at the thrust-block, are conservative.

The attempt to calculate the division of load between the cantilevers and the arches leads to tension in the up-stream face of the cantilevers, as it generally does unless the dam is quite high compared to its width. The authors do not accept this tension, but assume that cracks develop.

When a horizontal crack is produced it causes additional deflection of the cantilevers, which re-acts on the deflections of the arches and these, again, re-act on all the other cantilevers, thus causing a change in the division of load between cantilevers and arches. The writer has made a search of technical literature in order to ascertain how a deep horizontal crack, or several deep cracks, affects the deflection, since the ordinary theory is inadequate to deal with this problem. The only reference the writer has found[12] was one by Mr. B. F. Jakobsen, which deals with one crack only. This is a problem that is quite complicated and no claim is made to anything but a rough approximation.

The writer would like to know how the authors calculated the stresses and deflections produced by the cracks, and how they determined the location and depths of the cracks, because unless this can be done with some degree of accuracy the proposed method obviously fails or becomes a matter of pure guesswork. It would make considerable difference whether there were three cracks or five.

In Tables 1 and 2 the boundary stresses in the arches and in the cantilevers, respectively, are shown for a certain calculated division of load. For Cantilever *G* (Cantilever 27, Fig. 6), cracks are assumed to occur at various elevations. For intermediate elevations the depths of cracks are not given. To assume so many deep cracks is a rather astonishing practice. A crack 55.5 ft deep in a section 80 ft thick, leaving less than one-third the section intact, does not inspire any great confidence in the ability of the dam to act as a cantilever.

The writer has never seen such a variety of deep horizontal cracks in any dams, although he has spent much time in looking for them to study their cause, in order to avoid them in the future. He has found a number in various places in various dams, but never (or rarely) any definite horizontal cracks of any consequence. If they were present, such cracks would not be difficult to detect in dams provided with vertical drains near the up-stream face; they would also be indicated definitely with reservoir empty after they

[12] "The Cracked Cantilever in Dam Design," *Engineering and Contracting*, December, 1930, p. 439.

had closed. If as many horizontal cracks could be found on the up-stream face of the Ariel Dam as were assumed in Table 2, one would hesitate building the next dam.

The writer objects to a theory in which cracks are assumed as soon as apparent tension is encountered and in which the calculated depths of such assumed cracks are made the basis of deflection calculations and stress determinations. The stress distribution near such cracks has been disturbed to an unknown extent. One designer makes one guess, whereas some other equally competent engineer would make a different guess and would obtain different results. In the Stevenson Creek Dam, there was one horizontal crack at the bottom of the dam, but this structure is not of normal arch design and, therefore, not in the same class as the Ariel Dam.

On several arch dams, the writer has observed that the part of the arch between the quarter-points and the abutments, does not deflect in a down-stream direction as much as the deflection formula would indicate. An exaggerated case is the Stevenson Creek Dam, in which this part of the arch deflected up stream instead of down stream with full water load. This would indicate that the arch is loaded with more than the full water pressure toward the abutments. The deflection theory of arches as now available[13] does not take this phenomenon into account, as it cannot be explained theoretically, except perhaps as a buckling action.

The late Willis A. Slater, M. Am. Soc. C. E., stated[14] that on the Emigrant Creek Dam at Talent, Ore., "it was found that there were indications of an up-stream deflection" (of the arch toward the abutments). "With this double confirmation [Stevenson Creek Dam and celluloid model at Princeton University], there need no longer be any reason for doubting the indication of an up-stream deflection of the Emigrant Creek Dam." The Stevenson Creek Dam deflected 0.1 in, up stream toward the ends of the dam, with 60-ft head of water.

Ariel Dam has a crest thickness of nearly 20 ft and buckling would scarcely be expected. On this dam, however, both abutments moved radially, the south abutment 0.01 ft, which is twice that assumed in the design, and it is difficult to understand, why the deflection of Point *M*, Cantilever 36, Table 8, at Elevation 200 was only 0.008 ft (calculated to be 0.006 ft) unless some up-stream deflection of the arch at *M* relative to the abutment did take place, in which case the load on the cantilever is reversed. At Elevation 120, Point *M*, Cantilever 36, is located so close to the foundation and to the abutment that the arch could not buckle measurably up stream, and the measured deflection of 0.011 ft indicates that the foundation evidently has taken part in the down-stream deflection to a large extent.

The calculated deflection at this point, which is 0.003 ft, or $3\frac{2}{3}$ times smaller than the measured one, is supposed to create a horizontal crack 11 ft deep on the up-stream face. A much greater actual deformation would pre-

[13] "Etude sur les Barrages Arques," par Stucky.

[14] Rept. of Committee of Engineering Foundation on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 130.

sumably cause a much greater depth of crack. It is realized that such small deflections are quite difficult to measure accurately, but such errors in calculating the deflection must produce great errors in the division of load between cantilevers and arches.

For finding the stress distribution on horizontal planes in a cantilever, three assumptions can be made (one at a time, of course): (1) The vertical compression on horizontal planes is a linear function of the ordinate, x; (2) shear distribution on horizontal planes is parabolic; or (3) vertical stress is distributed according to the principle of least work.[15]

For a triangular profile or its equivalent, Assumption (1) gives the usual trapezoidal distribution with a maximum compression at the down-stream face. Assumption (2) gives a large vertical tension at the up-stream face, a large compression in the middle of the cross-section, and zero compression at the down-stream face.[16] Assumption (3) gives less vertical compression (or actual tension) at the up-stream face and less compression at the down-stream face than Assumption (1). The stresses obtained by the use of this method will average between those due to Assumption (1) and Assumption (2), and since less important assumptions were made in obtaining the design formulas for Assumption (3), these formulas express most nearly the actual stress distribution on horizontal planes near the foundation. Assumption (1) gives stresses at the up-stream face that are too favorable.

H. M. Westergaard, M. Am. Soc. C. E., has attacked the problem of arch design in a somewhat different manner.[17] He treats the structure as an elastic shell, rather than as a system of intersecting arches and cantilevers. Professor Westergaard advances the idea that when the influence of Poisson's ratio and of twisting moments is considered, the simple picture no more exists in which independent arches and cantilevers deflect equally at each point in common, when the load is divided properly between the arches and the cantilevers. He does not develop formulas, however, that can be used for calculating the stress, but states, that the present stage of the work is that of research and is too elaborate to be used directly in design. A designing engineer cannot be expected to have the mathematical ability of Professor Westergaard and, therefore, would have to leave the problem of the shell dam alone.

The late Fred A. Noetzli, M. Am. Soc. C. E., developed approximate formulas for the vertical arch action in curved dams,[18] which tend to demonstrate that a considerable part of the weight (in the example given by Mr. Noetzli, it is one-half the weight) of the middle high portion of the dam is transferred by means of vertical arch action to the hillsides. Only the remaining weight could then take part in resisting the water load through cantilever action. This is an important item to consider and it is not safe to overlook it, as has been done.

[15] "Stresses in Gravity Dams by the Principle of Least Work," by B. F. Jakobsen, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 489.

[16] "Technische Mechanik," by Otto Mohr, Second Edition, p. 296.

[17] Rept. of Committee of Engineering Foundation on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 231.

[18] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 36–39.

When designing an arch dam using, say, the Cain formulas for determining the stresses and assuming full water load acting on the arch, the experienced designer will always keep in mind that certain actions not directly taken care of by the simple Cain formulas (such as the spreading of the foundation, a temperature drop, and water-soaking on the up-stream face) tend to increase the tension at the intrados at the crown. The action of Poisson's ratio, however, tends to compensate for this tension, but at higher elevations this compensation is small.

Since it is necessary, for architectural reasons, to produce a smooth symmetrical appearing structure, it is impossible to keep the stresses uniformly as great as the maximum, or anywhere near this limit, at all elevations. Only at elevations where rather high stresses exist, would it be necessary to go over the calculations repeatedly, taking each unfavorable item into consideration separately. Tensile stress at the intrados at the crown is the most important stress on which to keep a check.

For lowering a high tensile stress at the abutments on the up-stream side, there are several compensating features not included in the formulas, such as water-soaking, foundation deformation, etc. That the compression should occasionally over-run the present allowable limit of 600 lb per sq in. is of minor importance.

It is of advantage for improved distribution of arch stresses that the Ariel Dam is provided with contraction joints spaced rather closely together, as at each joint the stresses have an opportunity to adjust themselves somewhat. In the past the writer has followed a tendency to space contraction joints rather far apart; this was with the object of saving in cost of joints. It will lower the actual maximum stresses, however, to have the joints close together (say, 30 ft apart in the zones of maximum stress) and to grout them with a mixture of cement and plaster of Paris, with or without a combination of chemicals, depending on whether the cracks are large or small. Plaster of Paris grout, used alone, expands 0.3% when setting. The combination is nearly neutral in this respect; the modulus of elasticity is much smaller than for the cement grout alone and will allow for some re-adjustment of the arch stresses better than the pure cement-grout.

The Ariel Dam section is thinner than it would have been, had the cantilever action been neglected. The same total yardage could have been obtained by increasing the arch stresses and neglecting the cantilevers as load carriers. The time saved in making such a design would equal the cost of many yards of concrete that could have gone into the structure instead.

As the technical theory stands developed to-day, with the necessity of making a considerable number of assumptions, in order to get anywhere it seems safest to depend on arch action only in arch dams or on gravity action only in gravity dams; but, even so, a combination of the two actions does occur, at least in some places.

H. B. MUCKLESTON,[19] M. AM. SOC. C. E. (by letter)[19a].—The writer has always felt some skepticism as to whether the results of computations by

[19] Cons. Engr., Vancouver, B. C., Canada.

[19a] Received by the Secretary September 29, 1933.

the trial-load method were worth the enormous labor involved. In the long run, the method is based on the elastic properties of the masonry and of the material in the foundations and abutments. It is certain that these properties are not uniform throughout the structure and, moreover, that they cannot be predicted with any certainty as to the facts. Hence, the behavior of the structure under load cannot be predicted with any certainty, either.

Tables 1 and 2 exhibit some predictions which are extraordinary, to say the least. For example, they predict cracks at Elevation 0, which are 56 ft deep extending two-thirds the thickness of the dam. The writer cannot accept these predictions as at all likely to be borne out by the facts. Is it a fact that the dam is cracked as the theory shows it should be? If not the theory is worthless. In Table 10 the writer has re-arranged Table 8. It

TABLE 10.—VALUES OF THE RATIO, $\frac{\text{MEASURED DEFLECTION}}{\text{CALCULATED DEFLECTION}}$

Elevation	POINT MEASURED (SEE FIG. 6 (*b*))							
	T	*A*	*C*	*E*	*G*	*I*	*K*	*M*
200.00	1.50	1.26	0.99	0.66	0.74	1.38	1.21	1.33
180.00	1.80	.....	0.86	0.71	0.86	1.36	3.00	3.70
120.00	0.26	.....	1.47	1.06	1.15	1.50	.....	.....
90.00	.....	.....	.....	1.20	.....	.....	.....	.....

will be seen that prophesy and performance do not agree by as much as 370 per cent. What is the use of carrying calculations to the fourth decimal place if facts may differ from predictions by any such amount as this? Even at Elevation 120.00, differences of 150% are shown.

If calculated strains may differ from actual strains by 150%, how far apart are the corresponding stresses? The theory indicates that either very high tension must exist in the up-stream face or, in the alternative, that cracks 56 ft deep have formed. This tension, or crack, is deduced from the deflection, as calculated, in the vertical plane. If a swelling of the masonry is assumed of only 1 part in 20 000, due to soaking, it will account for enough deflection to indicate nearly 100 lb per sq in. Poisson's ratio, assumed at 0.10, will account for nearly 50, the modulus being assumed at 2 000 000. If, as is much more likely, this modulus approaches 5 000 000, these two factors will account for considerably more.

If prophesy and performance differ by as much as this paper indicates they may, there is evidently something wrong with the theory upon which the prophesy is made

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DISCUSSIONS

EARTHS AND FOUNDATIONS PROGRESS REPORT OF SPECIAL COMMITTEE

Discussion

BY MESSRS. JOHN H. GRIFFITH, HARRY E. SAWTELL, AND O. K. FROEHLICH

JOHN H. GRIFFITH,[48] M. AM. SOC. C. E. (by letter).[48a]—In its consideration of the "Basic Problem" the Committee has sought to establish a fundamental dynamics applying to macroscopic matter on the basis of a molecular hypothesis. An arrangement of uniform cylinders or spheres is utilized in depicting the mechanical structure of the molecular system, and algebraic processes are used in deducing the analysis. Admitting the desirability of a general dynamics for earths a molecular theory seems futile in the light of history. In the past century the highly philosophical molecular theories[49] of Navier, Poisson, Boscovich, Aepinus, Poncelet, de Saint Venant and Lamé. have all been superseded by the potential algorithms of Green and others. The German authority, Sommerfeld,[50] has remarked that the matrix or summation method is simpler in principle, to which many will agree, but he adds that the wave analysis is simpler in practice. The idea of simplicity of procedure not only applies to molar and molecular matters, but always controls in business, finance, and industry.

The anomalies of the molecular theory are brought to view when it is considered with reference to the existing physical conditions. In Nature, energy is never diffused in space from a source through the angle of a pyramid, or at 60°, as premised. In general, it is diffused in three-dimensional space through the solid angle, 4π, and for the particular case of plane space through $2\pi = 360°$, as attested in the instances of heat, light, sound, and

NOTE.—The Progress Report of the Special Committee on Earths and Foundations was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in May, 1933, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: August, 1933, by Messrs. L. C. Wilcoxen, H. de B. Parsons, William P. Kimball, and T. A. Middlebrooks; September, 1933, by Messrs. Daniel E. Moran, and A. E. Cummings; and October, 1933, by Messrs. Edwin J. Beugler, Jacob Feld, George D. Camp, and Charles Terzaghi.

[48] Prof. of Experimental Eng., Iowa State Coll., Ames, Iowa.

[48a] Received by the Secretary, September 20, 1933.

[49] "Theory of Elasticity," by A. E. H. Love, Second Edition, pp. 7 to 11.

[50] "Wave Mechanics," by A. Sommerfeld, p. 37.

electricity. When energy is apparently "destroyed" or diverted by the agency of a grating, filter, reflector, pipe, wire, or insulator, as the case may be, the relative character of the phenomenon is to be apprehended. The diverting device is not to be regarded as an absolute non-conductor, but rather as the limiting aspect of a deformable medium. An insulator, in a similar manner, is regarded as a special case of a dielectric, etc. The corresponding dynamical analysis should correlate and should parallel precisely the condition imposed in the mechanical set-up in any rigorous accounting for the energy distribution. This is illustrated in the procedure of the English mathematician, Mitchell,[51] in his sketching and analyses of many distributions similar to those of the Committee as shown in Figs. 40 and 41. In each case he considers the distribution of stress in a cone under a load at the vertex. He knew that the general differential equations of dynamics apply to universal space distributions as in the instance of Laplace's well known equation,

$$\frac{\partial^2\phi}{\partial x^2} + \frac{\partial^2\phi}{\partial y^2} + \frac{\partial^2\phi}{\partial z^2} = \nabla^2\phi = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

whereas in particular the surface of the cone is free from stress and is surrounded by "blank" space (neglecting air or ether). He posited first the $4\text{-}\pi$ distribution for the whole of space. He then annulled the positive stress on the boundary of the cone by an appropriate opposing stress, giving the desired free boundary and resultant distributions both within and without to the surface. He chose the internal distribution to meet the needs and discarded the other.

Love[52] proceeded in a similar manner for the case of the Committee's basic problem remarking in his introduction[53] that the same fundamental dynamical principles are requisite and sufficient for the case of all physical science. To cite a particular case of classical dynamics where the untoward stress and energy is not equilibrated on a surface, but throughout a volume, there is the dynamical theory of polarization. Under normal conditions two opposing energies are equilibrated. For a suitable displacement of the plus and minus energies in the polarized condition, the so-called "pressure-bulbs" are generated precisely like those of Figs. 35 to 39, inclusive. Since there are only two energies in Nature—kinetic and potential—the attention of the civil engineer, in getting rational solutions, will be confined to the same essential methods as those of any other scientist in accordance with the principles of applied physics.[54]

Considering more specifically Love's [53] analysis for the "basic problem," the volume of pressure directly under the normal load on the infinite horizontal plane and co-axial with it (see Figs. 1 and 2), is found to be that for a cone of angular divergence, 68° 32′. The stress exterior to this cone is not zero,

[51] *Proceedings*, London Mathematical Soc., Vol. 32, 1900, p. 23, and Vol. 34, 1902, p. 134.

[52] "Theory of Elasticity," by A. E. H. Love, Second Edition, p. 188.

[53] *Loc. cit.*, p. 1.

[54] All these principles of dynamics are given in a precise compass in J. G. Coffin's "Vector Analysis"; the paper by J. G. Leathem, St. John's Coll., Cambridge, England, "Volume and Surface Integrals Used in Physics," gives a philosophical analysis, *Cambridge Tract No. 1.*

according to the Committee's analysis, but a field of tensile, pressure, and shearing stresses according to Love. The stress at the vertex of the model of Fig. 1 is undefined and not determinate as in the case of the second row of "molecules" taking one-half the load at the vertex. The stress actually should become infinite. This dynamical behavior is experienced generally throughout Nature for point loads, point electrodes, tapered conductors, and analogous mechanical set-ups exposed to similar fields. On the other hand, the function should be finite for conductors of finite area according to the analysis given later. The Stirling type of procedure has no dynamical significance and is merely a roundabout procedure for deducing Boussinesq's strictly rational analysis. Perry's elementary mechanics[55] is easier to follow than the elaborate details of the report.

It is probably correct enough, but appears a little uncommon to refer to "radiation of pressure from a load source" in the sense that stress is a manifestation of energy distributed radially into space as a result of a change of density against resistance. This type of radiation, however, is not realized in the Committee's model; for the 60° molecular inclines the distribution is radial; in general, the strain energy is transmitted through the "molecules" in many zig-zag lines composed of numerous small "tacks" pieced together. The distribution recommended by Love, on the contrary, is wholly radial, the resultant stress or "radiation" being,

$$R_z = \sqrt{Z^2_x + Z^2_y + Z^2_z} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

It is stated in the report that "the pressure distributions * * * are independent of the type of material and involve no elastic constants". This statement can only apply, as the writer remarks in a previous discussion,[56] to Love's expression,

$$Z_z = \frac{3\,P\,Z^3}{2\,\pi\,r^5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

It is not true for the vertical pressure in the Committee's molecular model. It is not true generally either for the Committee's or for Love's treatment, as, for example, in the special instances of the computation of lateral pressures and the stresses on planes other than the horizontal. The pressure distribution not only will depend on the dimensions of the loaded body, as shown experimentally by A. T. Goldbeck, M. Am. Soc. C. E.;[57] it depends as well upon the nature of the transmission material as defined by its several moduli.

The Committee posits "sufficient frictional resistance between the rollers to prevent lateral motion" without, however, providing for the mathematical law of transmission of friction. It ignores the possibility of cohesion. The late William Cain,[58] M. Am. Soc. C. E., showed that these factors should

[55] "Theory of Elasticity," by A. E. H. Love, Second Edition, p. 189; see end of Chapter 15 of Perry's "Mechanics" for simple treatment.

[56] *Proceedings*, Am. Soc. C. E., May, 1932, p. 778.

[57] "American Highways," Pt. IV, Chapter 1, p. 17, 1925.

[58] "Earth Pressures, Walls and Bins," by William Cain, 1916, see Chapters 1 and 5.

properly be taken into account in a theory of earth transmission, or otherwise provided for, as illustrated in his generalized Rankine theory and confirmed experimentally in the case of Cain's tests. In pulling a pile or a tree stump from the ground not only must frictional and cohesional resistances be considered but they must be considered in the case of matters such as clay, shale, or rock; obviously, the formula based on Stirling could scarcely differentiate the disparate behaviors which would occur, calling for moduli or other transmission coefficients. In fact, the tests considered in early reports are for this very purpose. The model will pull apart under an upward acting load suffering a mechanical discontinuity of the structure. Equation (47), on the other hand, is strictly applicable to the cases of either positive, negative, or imaginary loads, P. As long as there are no physical discontinuities experienced to modify the actual structure of the medium, Equation (47) deduced from Boussinesq's general potential theory,[59] while implicitly considering cohesion and friction avoids the necessity of an explicit mention of these factors. They are found deductively when desired.

To avoid the types of anomalies and incongruities cited, and others mentioned later, it is important that engineers should adopt in its entirety Boussinesq's algorithm from the classical dynamics for dealing with earth-resistance formulations rather than seek for molecular theories for this purpose created with respect to the algorithm to support the arguments, but having no scientific significance. In the case of a mixed or "mongrel" theory, part algebraic, part dynamical, it will be impossible in conformity with the doctrine of correspondence (Bohr) for the algebraic theory, or *vice versa,* to merge with the dynamical theory by a continuous variation of the functions. As an illustration, the Stirling algebraic type of formula (Equation (7) of the report) for example, cannot merge with Dr. Terzaghi's hydrodynamical type, such as Equatiaon (18), or Professor Gilboy's formula, $K \nabla^2 u = \frac{\partial u}{\partial t}$. In a true dynamical expression there is this merging in transitional cases. Einstein's theory merges with that of Newton. The equations of motion apply to rest when there is no acceleration; the theory of viscosity reduces to that of perfect liquids under appropriate conditions, etc. Boussinesq's formulations have this continuous variational property. His treatises and shorter papers in the *Comptes Rendus* des Academie des Sciences, Paris, give the logical analytical dynamics between Love's "Theory of Elasticity" and Lamb's "Hydrodynamics," complementing the theory of viscosity. As engineers long ago adopted the theories of de Saint Venant as applied to frameworks and beams, they cannot do better than adopt the logical continuation of de Saint Venant's analyses by his successor and most distinguished pupil.

All the problems of earth resistance involving infinitesimal strains are solved more directly and expeditiously by Boussinesq's dynamics of the

[59] Boussinesq's complete theory is contained in his "Application des Potentiels * * *," Paris, 1885, and the "Essai Theorique sur L'Equilibre des Massifs Pulverulents * * *," Bruxelles, 1876.

potential than by any other method, provided the earth is isotropic or quasi-isotropic. For the case of large strains, Taylor's expansion is the logical and only method of analytical continuation. Boussinesq[60] used the Maclaurin development, a special case of the Taylor development. His theory for large strains merges with the theory of elasticity in the limiting case, just as Rankine's formula for columns merges with the short and long-column formulas for appropriate values of the slenderness ratios, $\frac{l}{r}$[60].

Boussinesq's analysis will be applied to the cases of the report to illustrate the logical consistency, flexibility, and mathematical rigor. Referring to the Committee's Figs. 8, 9, 16, 17, and 35 to 39, inclusive, all the phenomena may be explained and formulated correctly in conformity with Boussinesq's analyses. The laws for stress and displacement along the axis in Figs. 8 and 9 are correct in form, but incorrect analytically. The same type of law applies to both since shears vanish along the axis. Boussinesq's expression for the displacement, w, is,[61]

$$w = -\frac{1}{4\,\pi\,(\lambda+\mu)}\frac{\partial\phi}{\partial z} - \frac{1}{4\,\pi\,\mu}\frac{\partial^2\psi}{\partial z^2} + \frac{\lambda+2\,\mu}{4\,\pi\,\mu\,(\lambda+\mu)}\nabla^2\psi \ldots\ldots(48)$$

in which, $\phi = \int\int p\ r\ dx_o\ dy_o$; $\psi = \int\int p \log (z+r)\ dx_o\ dy_o$; and r is the radius vector or diagonal to the stressed point, w, thus,

$$r = \sqrt{(x-x_o)^2 + (y-y_o)^2 + z^2}$$

measured from the point of stress application, x_o, y_o, z_o, on the surface of loading, z_o being zero when the origin is taken in the ground plane. The load stress, p, varies in intensity as an ellipsoid or paraboloid (nearly), the maximum being on the axis and the stress vanishing on the rim of the body where there is a adjusting flow or easement under the tendency to infinite stress under a sharp edge. For an easy engineering solution the compression modulus, λ, is taken equal to the shearing modulus, μ, corresponding to a Poisson's ratio, $\frac{1}{4}$, and the foregoing differentiation (Equation (48)) is conducted giving for the vertical component along the axis under the average stress, p_a:

$$w_a = p_a\left[\frac{5}{4\,\mu}(d-z) - \frac{1}{2\,\mu}\frac{(d-z)^2}{d}\right]\ldots\ldots\ldots\ldots(49)$$

in which, $d = \sqrt{R^2+z^2}$, R being the mean radius of the loading surface. A wider latitude of application is taken when desirable by letting p be variable and by using both moduli, λ and μ, the present treatment being sufficient for practice. In Equation (49), μ is to be determined by field tests to fit the observations. Separate tests give an order of 500 to 1 000 lb per sq in. for μ, depending on the type of earth.

[60] "Essai Theorique sur l'Equilibre des Massifs Pulverulents * * *," par. J. Boussinesq, p. 6 *et al.*

[61] "Theory of Elasticity," by A. E. H. Love, Second Edition, p. 190.

The foregoing solution agrees with all the observations for this case made to date. To illustrate the general value, w plats to the bell-shaped surface measured by Moyer and Fehr, Goldbeck, Enger, and others for both stress and deflections, the vertical cross-section being a generalized "witch", in the case of stress. The maximum value is on the axis under load. The surface vanishes asymptotically with the vector, r, becoming infinite on any horizontal plane, $z = c$, below the ground surface. When z is zero, the value of the loading stress is determined. When z becomes infinite, the stress is zero. The so-called "pressure-bulbs" are particular cases of the stress distributions defined[62] by Anson Marston, Past-President, Am. Soc. C. E., for plane space when the diameters of rollers or wheels become infinite. Such distributions[63] are characteristic of the general potential theory for solids and liquids. The contours of the Committee's Figs. 16 and 17 will ultimately smooth out to circles, in conformity with de Saint Venant's theory[64] of "statistically equipollent systems" and Boussinesq's theory of "perturbations locales" (local interference effects). The curves of Figs. 8 and 9 result directly from Equation (49). The correct right-hand equation of Fig. 9 is found when $R = 0$ for a point load. The correct left-hand curve is found when R is not taken as zero. For large strains, such as in the case of forest loams, sawdust, or "steel wool", Equation (49) may be taken in the manner of Boussinesq's expansions, or,

$$w_a = p_a \left[A_1 \frac{(d - z)}{d^0} + A_2 \frac{(d - z)^2}{d^1} + A_3 \frac{(d - z)^3}{d^2} \right] \ldots\ldots (50)$$

adding more terms if desirable and adjusting the constants, A, to suit. As already shown in an earlier discussion according to Winfrey[65] the method of integration is more exact than that of adding point loads. The series used must be convergent, but the equations of physics usually converge.

As several misunderstandings have arisen from time to time in the discussions relative to Boussinesq's formula, Equation (47), or, $p_z = \frac{3\,P}{2\,\pi\,z^2} \cos^5 0$, of the report, the following information will aid in its applications to the cases of different earth formations. The original form as submitted by the Bureau of Standards in 1917 to the Special Committee of the Society on the Bearing Value of Soils for Foundations, etc., was,

$$Z_z = K \frac{P z^3}{r^5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

the K being an empirical constant, 0.9, when applied to tests of Goldbeck on the sand-bin measurements of the U. S. Bureau of Public Roads.[66] In the report[67] of the Special Committee on Bearing Value of Soils for Foundations,

[62] *Transactions*, Am. Soc. C. E., Vol. XXXII, (1894), p. 273.
[63] "Newtonian Potential," by Benjamin Peirce, 1902, p. 187.
[64] "Theory of Elasticity," by A. E. H. Love, Second Edition, p. 187.
[65] *Proceedings*, Am. Soc. C. E., May, 1932, p. 779.
[66] *Proceedings*, Am. Soc. for Testing Materials, Vol. 17, 1917, p. 650.
[67] *Proceedings*, Am. Soc. C. E., August, 1920, Papers and Discussions, p. 932.

of 1920, the constant of Boussinesq was taken as $K = 0.48$, the volume of the bell-shaped surface of stress then being only about one-half the load, P. To meet the criticisms of this discrepancy and to obtain an exact rational formula to apply to different types of earths the resultant radial stress or "radiation", was taken in the form,

$$R_z = \frac{n-2}{2\pi} P \frac{\cos^{n-1}\phi}{z^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (52)$$

If this stress is projected on the element, $dx\ dy$, of the plane, $z = c$, in the directions, x, y, and z, for the pressure and shears, and these are substituted in the differential equation,

$$\frac{\partial Z_x}{\partial x} + \frac{\partial Z_y}{\partial y} + \frac{\partial Z_z}{\partial z} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (53)$$

it satisfies. Moreover, the shears balance among themselves and $\int z_z\ dx\ dy$ balances the load, P. The parameter, n, may be taken to fit the various experimental or analytical cases. For example, to fit Boussinesq's normal stresses and shears the value of n is 5. The value, 8, is required to fit Goldbeck's pressure curves. Other values would be needed to fit Housel's and Kögler and Scheidig's distributions. This type of surface is certain to fit closely with any tests of this character on account of the three conditions imposed on it, namely, satisfying the essential differential equation, always balancing the external load, P, and making the adjusting parameter, n. Moreover, the value, R_z, for the radiation will apply to both small and large strains, hysteresis being present, or being zero.

HARRY E. SAWTELL,[68] M. AM. SOC. C. E. (by letter).[68a]—This excellent report is one more step toward a better understanding and solution of many difficult foundation problems.

It would seem that many more experiments and many accurate determinations of the causes of settlement of structures must be made before a generally useful procedure can be developed, owing to the endless number of kinds of soils and conditions under which they are to be found and used.

The writer is much interested in the "Simplified Example of Pressure Distribution and Consolidation". The Committee states that the analysis is of an actual case of a large building erected upon the deposit of the Charles River Basin, near Boston, Mass.

Having had considerable to do with the design of this building, and noting that some of the assumptions as to the soils and foundations were not accurately stated, the following modifications are submitted with a feeling that such an important example should be correct in every way, to be of permanent value.

Referring to Fig. 14, the soils under the foundations are more nearly like the following: 10 to 22 ft of black muck and silt; 1½ to 13 ft of sand

[68] Associate with Charles T. Main, Inc., Cons. Engrs., Boston, Mass.
[68a] Received by the Secretary, September 23, 1933.

and gravel; 65 ft of clay, mostly soft; and 9 to 25 ft of boulder clay above the soft slate bed.

The Committee states further that "the building was placed upon piles driven into the clay * * *." The record of the pile-driving shows that more than one-half the heavily loaded piers, supporting the dome and sur-

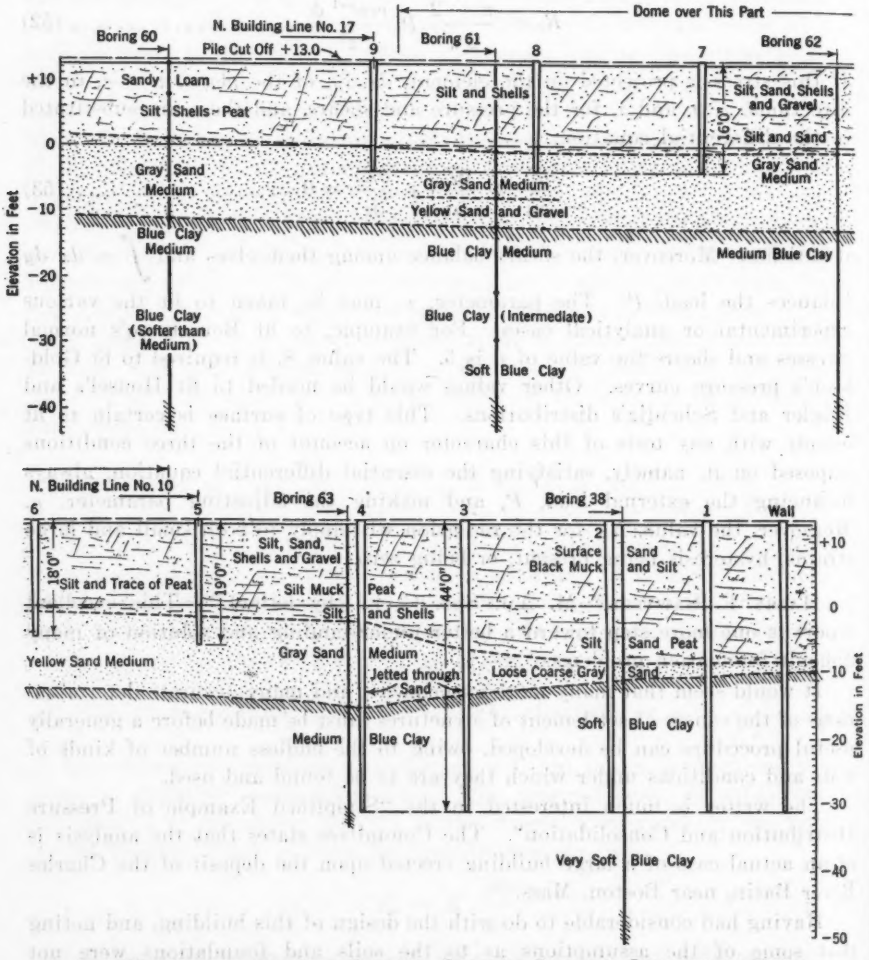

FIG. 63.—SECTION THROUGH BORINGS, SHOWING SOIL AND LENGTHS OF PILES

rounding structure, are carried by piles which were driven into, and stopped in, the glacial gravel and sand—never having reached the clay. These were the only piles to be driven to practical refusal. A vertical section through the soil and piles drawn on the long axis of the building is shown in Fig. 63.

The assumption made in the report is that driving piles into clay completely remoulds the clay for the depth penetrated by the piles, and such an

assumption was made in the case of this "Example". This assumption can not be correct for all kinds of clays, or for similar clays having different quantities of water content or different surcharge pressures and different hydraulic-head conditions. The writer doubts if it is true, even under the conditions noted in the "Example". The clay under this "Example" was found to be medium directly under the thicker parts of the glacial sand and gravel, but below that, it was soft, and very soft under all parts of the building.

Under the other buildings of this group, the clay was found to be medium, resulting in much less settlement. This building is not, therefore, "typical" as stated in connection with Figs. 16 and 17, but is special in this site. Referring to Fig. 17, it will be noticed that the settlements are quite uniform about the center of the dome structure (having large piers), being generally greater at the center, as would be expected. Keeping in mind that actually about one-half this structure is supported on piles resting in the glacial sand alone, and about one-half resting in the clay, it is interesting to note that the settlement diagram, Fig. 17, gives no indications of any difference in the character of supports.

If driving piles into the clay completely remoulded the clay around them, it is quite certain that one part of the building would have settled more than the other. In the original design of the foundations, a difference was made in the loading to be given piles which were stopped in the sand from those driven into the clay. Those in the clay were designed for about 40% less load than those in the sand. It was assumed that those resting in the sand would have the loads spread and distributed so that the additional load applied to the clay per square foot would be about the same under both, and thus would produce an equal settlement. Fig. 17 seems to confirm the real value of the original assumption.

The design of the piling did not allow for an additional weakness in the clay from the so-called "remoulding" effect. A large number of load and settlement tests, of about thirty days' duration, were conducted before the original design was made from which the different values for different conditions were assumed.

Another building was constructed recently upon this same site and a study was made of settlements of old buildings surrounding the new building. It was found that regardless of whether the piles were supported by the clay or by the sand and gravel, the settlements were about equal. The settlement of the buildings surrounding the new building was about $5\frac{3}{4}$ in., and all appear to have come to rest in less than the sixteen years of their existence; but these all have medium clay under them while the "Example" used in the Committee's report, had soft clay under it. All the pile tests for the new building, including re-driving tests and load test, show quite clearly that no destructive remoulding took place in the medium blue clay found at this site.

Regarding the stress distribution in soils by the equations of Boussinesq, the writer would state that it is difficult to understand how the results can be at all accurate, as it has been stated that the equations apply only to the elastic solids having the same properties in all directions (isotropic).

The manner in which clay is constructed in Nature, together with its consolidation by an overburden, produces laminations and construction which gives it different properties vertically and horizontally. Most clays have large percentages of sand and this is not distributed equally by any means, but often is found in lenses or pockets.

It is difficult to understand how any equation can be used with a satisfactory degree of accuracy, but the study of soils and their properties will be very profitable.

DR.-ING. O. K. FROEHLICH[69] (by letter)[69a].—The example in Fig. 1 of the report can be shown to give Equations (1) to (6) only if the balls, or rollers, are replaced by tiers of beams (See Fig. 64(*b*)). The rollers and balls give radial components which are neglected in the example cited and which alter the pressure distributions. Frictional resistance alone cannot displace these radial components. An infinite number of pressure distributions is possible in a stack of rollers, the limits of which are shown in Fig. 65. For this reason the introductory examples are not wisely chosen; they do not show the radial spread which is particularly characteristic of pressure distribution in soil.

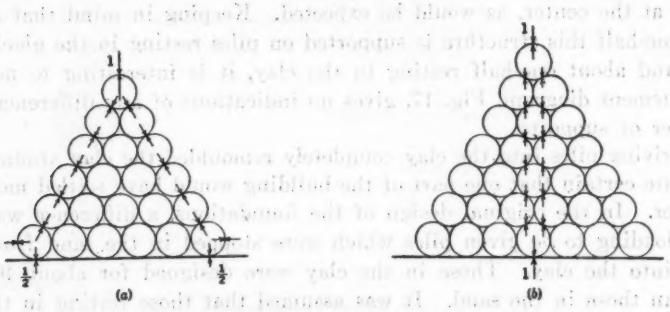

FIG. 64.—EXAMPLES OF PRESSURE DISTRIBUTION

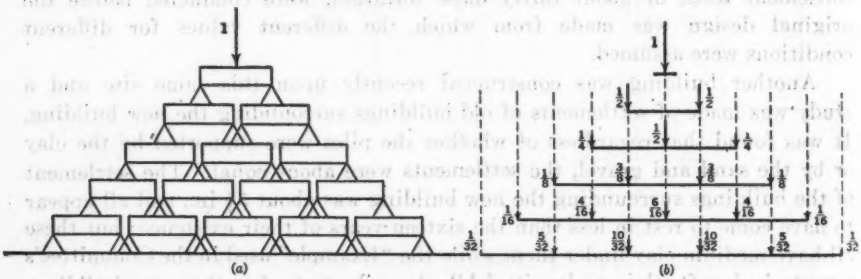

FIG. 65.—PRESSURE DISTRIBUTIONS IN SIMPLE BEAMS AND IN A SOLID MASS

Referring to Figs. 4 and 5 of the report it is stated that "the assumption of a load concentrated at a point makes it necessary to go some distance below the point of application of the load to obtain fairly accurate results."

[69] The Hague, The Netherlands.

[69a] Received by the Secretary, August 2, 1933.

The writer will attempt to derive expressions to determine the limits of the vague expression, "some distance." Assuming that the Committee is correct in considering Boussinesq's law valid for the vertical normal pressures in the range of "some distance" from the load to infinitely great distances, it would be possible to approach closer and closer to the load, until, at a certain distance, one suddenly finds that elastic equilibrium is no longer possible. The limiting condition for a soil in which the cohesion factor, p_k, is constant and in which the angle, ϕ, represents internal friction, can be expressed by the following formula:

$$\sigma_1 - \sigma_3 = 2\,k\left[p_k + \frac{1}{2}\,(\sigma_1 + \sigma_3)\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

in which, σ_1 and σ_3 are the extreme principal stresses at any point of the mass and k $(= \sin \phi)$ equals a constant value. Equation (54) defines the condition of plasticity according to Mohr. In the case of a loose granular mass, if $p_k = 0$,

$$k = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

Equation (55) might be termed "Rankine's" condition of plasticity for a loose granular mass because it plays an important part in the theory propounded by that scientist.

Finally, there are materials, such as metals, noted for high values of p_k and low values of k. For such materials, the second term within the bracket in Equation (54) may be omitted:

$$\sigma_1 - \sigma_3 = 2\,k\,p_k = \text{constant} \quad \ldots\ldots\ldots\ldots\ldots\ldots (56)$$

Equation (56) expresses the theory of plasticity in its most basic form as derived by de St. Venant.[70] Neither Equations (54), (55), or (56), contain the second principal stress, σ_2, defined by the condition, $\sigma_1 > \sigma_2 > \sigma_3$. In recent years, von Mises[71] and Schleicher have introduced a more general condition of plasticity in which the intermediate principal stress, σ_2, plays a certain part.

For present purposes Mohr's condition (Equation (54)) seems to be sufficiently general. Consider a vertical concentrated load, P, acting at a point on the surface of an infinite mass with given values of p_k, k, and γ. Let the vertical downward line in which P acts, be the axis, and let its point of application be the pole of a system of polar co-ordinates r, ζ. At any depth, z, below the surface, the three principal stresses may be $\gamma\,z$. After applying the load, P, the three stresses at the point, r, will be expressed by,

$$\sigma_1 = \frac{3\,P}{2\,\pi\,r^2}\cos\zeta + \gamma\,z \quad \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

$$\sigma_2 = \sigma_3 = \gamma\,z \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (58)$$

[70] "Plasticity, a Mechanics of the Plastic State of Matter," by A. Nadai, N. Y. and Lond., 1931.

[71] "Mechanik der plastischen Formaenderung von Kristallen," *Zeitschrift* für angewandte Mathematik und Mechanik, 1928.

The first term of Equation (57) is the correct polar principal stress acording to Boussinesq and Michell for a material with Poisson's ratio equal to 0.5, which is equivalent to constancy of volume. Combining the condition of plasticity as given by Equation (54) with Equations (57) and (58), and using the expression, $z = r \cos \zeta$, a curve equation results between r and ζ defining the zone within which plastic flow takes place and for which region the Boussinesq equations are no longer valid. The equation is,

$$\frac{3\,P}{2\,\pi\,r^2} \cos \zeta = k \left\{ 2\,p_k + \frac{3\,P}{2\,\pi\,r^2} \cos \zeta + 2\,\gamma\,r \cos \zeta \right\} \ldots\ldots\ldots (59)$$

The value of r for which the law is no longer applicable is a function of the properties of the soil, the amount of the load, P, and the angle, ζ. Most experiments have been made in cohesionless sand ($p_k = 0$); the condition expressed by Equation (59), with this value, becomes:

$$r = \sqrt[3]{\frac{3}{4} \left(\frac{P}{\pi\,\gamma}\right) \left(\frac{1-k}{k}\right)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (60)$$

In Equation (60), r is the radius of a hemisphere with the point of application of P as the center, within which plastic flow takes place.

In similar fashion a plasticity condition equation for the two-dimensional problem with surface load, p_o, may be derived, and a limiting value, p'_o, may be found which the surface load, p_o, must not exceed if valid readings are to be found near the surface. This value is,

$$p'_o = \frac{\pi\,p_k}{\cot \phi - \left(\frac{\pi}{2} - \phi\right)}$$

In Fig. 8 of the report the isobar line for $P = 1$ is incorrectly drawn, as no points of inflection exist in the equations. The question of the determination of valid readings within the plastic zone has been discussed by the writer elsewhere.[72]

Referring to Equation (18) of the report, a dimensional equation shows that the coefficient of consolidation may be outlined as $c = \frac{cm^2}{\text{sec}}$. The correct definition, dimensionally, of the coefficient should read "area divided by time" and not, as expressed in the report, "centimeters per minute," or dimensionally, "length per unit of time." Equation (15) of the report should, therefore, be replaced by the following: $c = \frac{k}{\gamma\,a}(1 + e)$, in which, γ denotes the unit weight of the liquid filling the voids of the clay (water); all symbols in this definition equation are to be expressed either in "true" or in "reduced" units of length. Fortunately, the determination of c is not generally based on this equation, but on Equation (33), of the report, which is correct if h_o represents one-half the reduced thickness of the clay sample.

[72] "Drukverdeeling in bouwgrond," *De Ingenieur*, 1932.

However, in order to obtain identical results, when using the two possible methods of determining the value of c, the foregoing corrections of Equation (15) are necessary,

The computation of the true settlements appears to be unaffected by the use of either true or "reduced" lengths, which may be shown as follows: Using the true value, h, the "true" settlement, S'_t, equals,

$$S'_t = Q\,h\,\frac{e_i - e_f}{1 + e_i}$$

In reduced units, the settlement, $S''_t = Q\ h_o\ (e_i - e_f)$. However, since $h_o = \dfrac{h}{(1+e_i)}$ then, $S'_t = S''_t$, confirming the conclusion as to the equivalence of the computations in true or reduced units of length.

The complete process of consolidation is well demonstrated by the voids-ratio-pressure diagram in which the load, p, on the clay sample, equals the vertical principal stress, σ_1. No reference was made in the report concerning the remaining two principal stresses, σ_2 and σ_3, both of which are horizontal. Because of symmetry, σ_2 may be written equal to σ_3. A means of determining the horizontal principal pressure, σ_3, has been published elsewhere, [73] in which the ratio, $K = \dfrac{\sigma_3}{\sigma_1}$, is introduced having a value 0.70 for yellow residual clay and 0.75 for blue marine clay. This factor, K, termed the "coefficient of the pressure at rest," can be determined from the constants of elastic isotropic materials as follows:

$$K = \frac{1}{m - 1} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (61)$$

in which, m is the reciprocal of the Poisson ratio. (A value, $m = 2$, denotes an incompressible material.) Applying Equation (61) to the aforementioned clay samples, $m = 2.428$ and 2.334, respectively, differing comparatively little from the value, $m = 2$, which may be taken as a proof that the pressure distribution in the elastic part of the clay at some distance from the load must follow, approximately, Boussinesq's law.

The consolidation of clay is sometimes referred to as a phenomenon of plastic flow. It should be noted, however, that by the theory of plasticity plastic flow is defined by a condition of plasticity. In the case of clay it is necessary to distinguish two important limiting cases, namely, (1) the case in which a load is being applied very suddenly so that consolidation, practically speaking, has not begun at the time the load is removed (as in test loads); and (2) the case in which the load is applied quite slowly and during a relatively long period, so that consolidation may take place (as in tests by means of consolidation apparatus). It can be shown that the condition representing plasticity for Case (1) may be written,

$$\sigma_1 - \sigma_3 = \frac{2\,k\,p_k}{1 - k} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (62)$$

[73] *Engineering News-Record*, Vol. 95, No. 20, November 12, 1925, p. 798.

and for Case (2),

$$\sigma_1 - \sigma_3 = 2\,k \left\{ p_k + \frac{1}{2}\,(\sigma_1 + \sigma_3) \right\} \ldots\ldots\ldots\ldots\ldots (63)$$

Equation (63) is identical with Equation (54). It is quite certain that consolidation is a plastic phenomenon, although, at first glance, it seems difficult to explain it in terms of the existing theory of plasticity. Nevertheless, it can be shown that the condition of plasticity expressed by Equation (63) is applicable to the action of the hydrodynamic excess, $w = p'' - p'$, and leads to a state of equilibrium in the solid clay mass defined by the factors, p_k and ϕ, the cohesion, and the internal friction, the voids-ratio being reduced, in the course of the plastic flow, from e' to e''. Equation (62) can be applied to foundation problems, in which consolidation in a vertical direction and lateral flow of the clay occur simultaneously. A practical example of this combined process is presented as Case *F* in the report of the Committee, in which the difference between the computed settlement curves and the curves actually observed is explained by referring to the influence of the lateral flow of the soil. The criterion for lateral flow under an evenly distributed load can be represented by the following equation,

$$p_o \text{ (critical)} \gtrless \frac{\pi\,(p_k + \gamma\,z_o)}{\cot\phi - \left(\frac{\pi}{2} - \phi\right)} \ldots\ldots\ldots\ldots\ldots (64)$$

in which, γ is the unit weight of the soil; p_k is the cohesion of the clay layer (weight per unit area); z_o the depth of foundation; and ϕ, the angle of internal friction. Equation (64) shows that the danger of lateral flow varies inversely as the sum of the cohesion and the original pressure in the soil at the level of the foundation.

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DISCUSSIONS

USE AND CAPACITY OF CITY STREETS

Discussion

BY MESSRS. R. O. THOMAS, SIGVALD JOHANNESSON, W. W. CROSBY, AND HAROLD M. LEWIS

R. O. THOMAS[26], ESQ. (by letter)[26a].—A timely emphasis of the fact that city streets are not used, either economically or actually, to their greatest capacity, is afforded by this paper. The writer agrees with Mr. Simpson's data and conclusions in all respects except as to the influence of curb parking on street capacity.

Some of the results of a survey, in 1932, of the parking situation in the Central Business District of Los Angeles, Calif., may be of value in determining the extent to which street capacity is reduced by permitting parking in the lane next the curb. In this District about 37.7% of the total curb length is available for curb parking by private machines. Computed on a basis of 19.5 ft per car, there is room to park 31 000 cars at the curb during a business day. The hours during which parking is permitted for a 45-min period are from 8:00 A. M. to 4:30 P. M. This gives 31 000 ± car-periods. The writer is aware that 18 ft per car is the generally accepted curb length assigned to each car, but a series of about 1 500 measurements made directly at the curb, gives an average of 19.5 ft of curb space per car.

By actual count, the number of car-periods utilized at the curb during the day was 29 000. This discrepancy is caused by the fact that on the outskirts of the Central Business District, the curb is not utilized to its full capacity. However, by actual count, the number of cars parked, was 20 000±. These 20 000 cars, due to the fact that 5 000±, or 25%, remained overtime for one or more periods, used the space which might have been occupied by 29 000 cars parked for the legal time limit.

During the progress of the parking survey (which extended over a period of several months) attention was given to the utilization of off-street parking facilities, with results as shown in Table 15.

The turnover in each case was found by contacting the proprietor and employees of the parking garage, station, or lot. When a car is parked in a

NOTE.—The paper by Hawley S. Simpson, Assoc. M. Am. Soc. C. E., was published in August, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[26] Los Angeles, Calif.

[26a] Received by the Secretary August 28, 1933.

garage, it is assumed that it is the intention of the owner to leave it there throughout the day. That leaves the public and private open-air lots to care for the transient trade. The writer will concern himself only with the public parks. It is believed, and on good authority, that a daily turnover of 5.0 is

TABLE 15.—OFF-STREET STORAGE CAPACITY AND USE, CENTRAL BUSINESS DISTRICT, LOS ANGELES, CALIFORNIA.

Facility	Number	Capacity	Estimated turnover	Rate of turnover
Public garages	99	14 065	12 272	0.872
Private garages	71	5 461	5 181	0.948
Public automobile parks	306	32 769	40 041	1.224
Private automobile parks	117	3 998	2 531	0.632
Totals	593	56 293	60 025	1.068

not unreasonable to assume and that this figure can be easily reached by the average parking lot. Thus, with a capacity of 33 000, the parking lots could be expected to care for 165 000 vehicles per day.

The writer has gone into this subject at such length in order to provide a basis for the claim that it is not absolutely necessary, as it is frequently held, to destroy the capacity of the city streets in order to extend privilege to a favored few. It appears from the foregoing that the 20 000 cars which actually park at the curb during the average day could be adequately cared for by the parking lots. It was observed that although the total daily turnover was slightly more than the capacity of the lots, there was no time during the day when the lots were filled to anywhere near their rated capacity. The useless waste of space is tremendous.

In the Central Business District of Los Angeles, the streets are from 60 to 80 ft wide. If parking is allowed on two sides of the street, the effective width is cut to from 40 to 60 ft. Then, if a right of way is provided for two lines of street-car tracks the following situation arises: At any time a street car is traversing a block, it is impossible for automobiles to pass it because they cannot get between the row of parked automobiles on the one hand and the street car on the other. It is manifestly impossible for them to pass the street car on the left, because of the danger of collision with machines using the other side of the street. It appears, therefore, that when street cars are in a block on which parking is allowed, the private machines also using that block are necessarily limited in speed to the rate of travel of the street car. Consequently, Mr. Simpson's estimate of 5% as the reduction of capacity of the lane next the line of parked cars is too low; it should be 25, 30, or even as much as 40 per cent. Investigation along this line should prove extremely interesting.

The foregoing arguments are powerful ones for the abolition of parking in city streets. Such parking can be defended on no other basis than that of special privilege; it lessens the use of the streets to an alarming extent; it is not necessary, since parking lots are capable of caring for the relatively small percentage (about 20% of the total number of cars parked) of cars

parked at the curb; and, finally, the privilege is gravely abused, since out of a possible usage of 29 000 car-periods, only 15 000 cars, or slightly more than 50%, obey the law and park only for the legal length of time.

The remainder of Mr. Simpson's conclusions are of untold value and should be consulted by every legal body prior to the drafting of a traffic ordinance.

SIGVALD JOHANNESSON,[27] M. AM. SOC. C. E. (by letter)[27a].—It is evident that the author has made a careful study of available evidence in preparing this paper. An important statement in it is that "the passenger capacity of a city street is of much greater social importance than its vehicle capacity."

The indication is that the passenger capacity of a street is far greater when street cars are used for transportation, than when the passengers are transported by automobiles. The writer agrees with this in principle, although he may not agree with all the statements and assumptions made by the author.

In the matter of passenger capacity, Mr. Simpson has considered automobiles, buses, and street cars. It might be worth while in this connection to consider, also, the bicycle. The "operating area" of a bicycle, using the author's method of determining this area, may be taken to be $\frac{5\,280 \times 8 \times 3}{5\,280}$ = 24 sq. ft, as compared with 69 sq ft for the street car. On this basis the passenger capacity of a street would be nearly three times as great when bicycles are used for transportation as when street cars are used for this purpose. The question remains, however, whether the general public would favor the use of bicycles in city streets to the complete or partial exclusion of automobiles, just as the question remains whether it favors the use of street cars to the complete or partial exclusion of automobiles.

The title of the paper implies that only capacity is discussed. For a complete study of the economics involved in street transportation, it is necessary also to consider the time element.

Outside his working hours the value of a man's time is that which he himself chooses to place upon it, according to his desire and ability to pay. If he does not consider his free time to have any money value, then, within certain limits, the most economical means of transportation is walking. This means of transportation was once quite common, and is still largely used in places where ready cash is scarce. The reason why walking as a means of transportation, say, within a radius of 3 or 4 miles, is not generally accepted as formerly is principally, that people prefer to save transportation time and are able and willing to pay for this saving. For example, a man living within 2 miles of his place of business could walk this distance in 30 min, but prefers to take a street car, in which he can cover the distance in 10 min, saving 20 min for an outlay of perhaps 7 cents for carfare. Under these circumstances he considers, apparently, that the value of his own time is at least $\frac{1}{3}$ cent per min.

[27] Designing Engr., State Highway Comm., Trenton, N. J.

[27a] Received by the Secretary September 13, 1933.

As a rule, the general public places a money value on time and for that reason desires reasonably fast transportation and is willing to pay to obtain it. Quite properly the author states that the presence of street cars reduces the rate of speed at which automobiles can be operated in city streets, and an estimate may readily be made of the money value of the time lost due to this reduction of speed. The important question, however, is not whether street cars, buses, or automobiles afford the greatest passenger capacity, but whether the general public prefers a large "passenger capacity" rather than speed of travel. The former can be obtained most readily by walking or bicycling, and the latter by the use of automobiles unimpeded by the presence of street cars.

W. W. CROSBY,[28] M. AM. SOC. C. E. (by letter)[28a].—The writer has found this paper extremely interesting, and he considers it valuable for careful study and reference. One of its great merits lies in its suggestions of ideas for further discussion and development.

In the "Synopsis," Mr. Simpson points out that street improvement projects as well as traffic regulation plans should be developed so as to facilitate the movement of private as well as public vehicles to the greatest possible extent. Later, under the heading, "Introduction," he defends his view that the passenger capacity of a city street is of much greater social importance than the vehicle capacity, defining precisely his concept of the meaning of "capacity" under the heading, "The Meaning of Street Capacity."

American views have always been toward freedom of action of the individual, including probably the individual in his motor car. Whether the National Industrial Recovery Act (NRA) will bring about a permanent change of this ideal remains to be seen. For the present, however, ideas concerning passenger and vehicle rights, such as the preceding, invite careful consideration as to their general acceptance, with or without modifications.

Among the "Factors Affecting Capacity of Multiple-Lane Surface Streets," Mr. Simpson mentions bus traffic as a major one. This brings to mind that one can walk up Fifth Avenue, in New York City, from 30th to 55th Streets, or beyond, more quickly than one can travel by bus. In this case, the effect of the buses on the other traffic seems to be greater than that estimated by the author. The allowances suggested for the reductions in capacities by turns and by parking are at least conservative and perhaps should be doubled.

Mr. Simpson under-estimates the reduction of street capacities for motors by the presence of street cars under ordinary conditions. A statement that deserves emphasis is (see "Measures for Traffic Relief") that when the time comes for considering new street facilities, ordinarily, new and widened streets are offered as the solution. It is true as the author points out that the provision of added street areas for automobile use only often fails to provide the needed traffic relief; but in that case the writer does not believe that provisions for "mass transportation" are always indicated.

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[28] Cons. Engr., Coronado, Calif.

[28a] Received by the Secretary October 9, 1933.

The writer agrees, generally, with the advanced conclusions, but would suggest, with respect to the last sentence under "Conclusion," that the money expended should be in proportion to the social and recreational importance, as well as the economic importance, of each type of vehicle in the city plan. Even when considering large cities, it must be admitted that people live in them, who do not merely move from homes to work and back.

HAROLD M. LEWIS,[29] M. AM. SOC. C. E. (by letter)[29a].—This paper should be of great assistance to city engineers and city planners in arriving at a logical design of the roadways in city streets. There has been considerable confusion as to the capacity of streets and highways in terms of the number of vehicles they can carry. This has resulted largely from the use of the term, "vehicles per hour," as the designation of traffic capacity whether the figures used actually represented those passing within an hour or the average rate of vehicles within either shorter or longer periods. Much of this confusion would be eliminated if this term were not used when it was desired to express the intensity of traffic for periods of less than one hour.

Some of the lane capacity curves in Fig. 1 are based on the assumption that it would be possible to have traffic throughout a full hour of the intensity that has been found for periods of only a few minutes. It is just as illogical to consider such short period intensities as representing a possible hourly street capacity as it is to consider the tops of the waves the theoretical level of the ocean. Traffic also moves in waves, and he who is trying to determine the necessary width of a roadway should design it for something less than the intensity of a maximum wave of traffic. It is no more possible to fill up the space between these waves of traffic and still maintain rapid rates of speed, than it is to raise the ocean to the level of the tops of the waves.

The maximum number of vehicles that may reasonably be expected to pass a given point within one full hour is a reasonable basis for determining the width of a street or thoroughfare. At special locations, such as busy intersections and at the approaches or exits to bridges or tunnels, those intensities which may be found over shorter periods should be made a basis of local design. The maximum number of vehicles that might be expected within a 15-min period should furnish a basis for such designs.

Curve 9 in Fig. 1 was developed by the Regional Plan of New York and Its Environs, and is lower than the others because it was based on traffic that may be expected within a full hour and because of the more general use of four-wheel brakes since it was developed. It was presented in the Regional Plan report on "Highway Traffic"[9] as "a basis for designing future streets and checking existing capacities" in a single lane of traffic, and it was pointed out that a roadway with two or more lanes in each direction would have lower average lane capacities. The curve was based on observations made in 1924 of spacing between groups of vehicles. The universal use of four-wheel brakes since that time has made it safer for vehicles to drive closer together

[29] Cons. Engr.; Engr., Regional Plan Assoc., Inc., New York, N. Y.

[29a] Received by the Secretary October 18, 1933.

[9] Regional Survey of New York and Its Environs, Vol. III, "Highway Traffic," 1927, p. 86.

at the higher speeds, which would tend to decrease the drop in this curve after it reached its peak. Such a revision would make it somewhat similar in general shape to Curves 7 and 8, in Fig. 1.

Mr. Simpson has given a value of 900 vehicles per lane per hour as the capacity of a traffic lane devoted to straight-through movement on a surface street. While it is undoubtedly possible for traffic to reach such values in isolated cases, a better figure to use for the purpose of street design is 750 vehicles per lane per hour on streets with two or three moving lanes in each direction. This is 50% of the maximum capacity of a single lane on an express highway, which the writer agrees with Mr. Simpson is about 1 500 vehicles per hr.

During October and November, 1924, the Regional Plan Staff made a total of 983 observations of the volume and speed of vehicles on typical streets in the Borough of Manhattan. The velocity, including necessary stops for traffic, was found to vary from 2 to 25 miles per hr. The weighted average of all the observations was 11.3 miles per hr, and it was found that with this velocity many of the streets were carrying only 150 to 500 vehicles per moving lane per hr.

In 1932, the Regional Plan Association made some traffic counts to determine the number of persons and vehicles entering Manhattan south of 59th Street on a typical business day. In this connection a series of counts of persons per vehicle were made. They may be of interest as supplementing

TABLE 16.—NUMBER OF PERSONS PER CAR IN VEHICLES, EXCLUDING BUSES, ENTERING MANHATTAN SOUTH OF 59TH STREET ON A TYPICAL BUSINESS DAY IN JUNE, 1932.

Location	In passenger cars	In trucks
On Avenues between 58th and 59th Streets:		
First Avenue	1.62	1.42
Park Avenue	1.64	1.35
Fifth Avenue	1.61	....
Seventh Avenue	1.94	....
Ninth Avenue	1.29	....
Tenth Avenue	1.41	1.43
Columbus Circle	1.53	1.34
On upper roadway of Queensboro Bridge	1.69	....
Average observations crossing 59th Street	1.59	1.39
Holland Tunnel	1.88*	1.43*
From 23d Street Electric Ferry, across Hudson River	1.62*	1.52*
From 42d Street Ferry, across Hudson River	1.81*	1.58*
Adopted as a general average	1.7	1.4

* An average of several half-hour observations.

Mr. Simpson's figures. Such counts were made over half-hour periods and included the drivers of the cars in each case. The results are given in Table 16.

This same traffic count, together with statistics regularly kept for traffic on the ferries and bridges to Manhattan, has made it possible to compile figures for the City of New York comparable with those quoted in Mr. Simp-

son's Table 14, showing modes of transportation on city streets to and from central business districts. It was found that of all persons entering Southern Manhattan across 59th Street, excluding railroad passengers to the Grand Central Terminal, 74.0% used mass transportation vehicles (buses, trolleys, or rapid transit) and 26.0% used other motor vehicles (private cars, taxicabs, or trucks).

TABLE 17.—PERSONS ENTERING MANHATTAN SOUTH OF 59TH STREET, NEW YORK CITY, ON A TYPICAL BUSINESS DAY, IN JUNE, 1932

Means of transportation	Number	Percentage
By mass transportation vehicles (buses, trolleys, or rapid transit)	1 976 419	75.1
In other vehicles (private cars, taxicabs, and trucks)	494 408	18.8
Pedestrians on bridges and ferries	159 805	6.1
Total on city streets	2 630 632	100.0
By railroad	161 180	....
Total entering Southern Manhattan	2 791 812	....

The modes of transportation used by all persons entering that part of Manhattan south of 59th Street, from all directions, on a typical business day are as shown in Table 17.

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DISCUSSIONS

WATER-POWER DEVELOPMENT OF THE ST. LAWRENCE RIVER

Discussion

BY MESSRS. THERON M. RIPLEY, W. S. LEE, FRANK E. BONNER, RUFUS W. PUTNAM, JAMES W. RICKEY, WALTER M. SMITH, J. W. BEARDSLEY, AND L. F. HARZA

THERON M. RIPLEY,[16] M. AM. SOC. C. E. (by letter)[16a].—In considering this paper the reader is left in almost as confused a state of mind as has been the condition in the past from the reading of the various estimates put out by the Government engineers and other engineering organizations. Moreover, all these various reports and papers are devoted principally to the estimated cost of the construction of one or other of the various plans, and little or nothing is said about their economic justification. There has come to the writer's desk only one of these numerous reports that contained much information on this question of economics and in which the statement seemed to be based on estimated figures of costs of power delivered where there might be some probability of its use.

The "Report of the St. Lawrence Power Development Commission; Submitted to the Governor and the Legislature of the State of New York, January 15, 1931," contains an estimate of the cost of steam generated in, and of St. Lawrence power transmitted to, New York and up-State cities. It contains, also, extensive tabulations of the cost of electricity now furnished to many up-State municipalities and to New York City.

The Marketing Board, after submitting the aforementioned estimates and tabulations, completes its report to the Commission with these words:

"A further source of electric development appears in railway electrification. It seems probable that within a ten-year period enough of the railroads of the State will be electrified to require 1 000 000 000 kilowatt-hours per year.

NOTE.—The paper by Daniel W. Mead, Hon. M. Am. Soc. C. E., was presented at the Joint Meeting of the Power Division of the Society with the American Institute of Electrical Engineers and the Hydraulics Division of the American Society of Mechanical Engineers, Chicago, Ill., June 29, 1933, and was published in August, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: August, 1933, T. H. Hogg, M. Am. Soc. C. E.

[16] Cons. Civ. Engr., Buffalo, N. Y.

[16a] Received by the Secretary September 9, 1933.

"In view of the past increases in power required and in view of the special developments that are likely to take place, we conclude there will be an ample market for St. Lawrence power and that there will be sufficient continuous load to take the entire St. Lawrence output available to New York within a brief period after the completion of the plant."

There is no doubt but that contracts for a large portion of all of the St. Lawrence power could be secured providing said contracts were on a basis of guaranteed delivery with the resources of the State of New York behind such guaranty.

With the knowledge that the report on the New York Plan was made by a Commission appointed by a Governor who was known to be sympathetic to the proposition of the State developing its water powers, and, further, in view of the fact that the Commission was careful to confine itself to the realm of possibilities rather than probabilities, one is still left in doubt as to whether the findings should be accepted as conclusive economically or whether there were other political and social reasons for the conclusions stated.

In other words, the question is: Can electricity be furnished to "possible" users for a less expenditure of time and money than by the development and transmission of St. Lawrence power?

During the three decades that this problem has been a subject of widest discussion the cost of producting electric power by steam has undergone radical changes in the following major lines: (*a*) The development of steam turbines; (*b*) the development of the mechanically fed boiler (which is best illustrated by the change from a grimy man with a coal scoop to the white-collar man in front of an instrument board); (*c*) the construction of the high-speed generator with a capacity of 100 000 hp; and (*d*) the transmission of the power at voltages never dreamed of at the beginning of the Twentieth Century. In spite of all these changes, power development on the rapids of the St. Lawrence still holds a position in the public conscience which the writer believes is due in a large measure to the magnitude of the undertaking.

In Tables 1 and 2, Professor Mead shows operating charges and net income. The former is based on a capacity factor of 100% and, therefore, the power would have to be used at the point of generation continuously throughout the year. Justly, the author was not concerned with the economics of power use; nevertheless, this question must be a main issue at all times. It is believed that no one is so optimistic as to believe that 1 000 000 hp, even if available, would be used along the banks of the St. Lawrence River in New York State within any time that can be predicated. The moment this power is put on the transmission line its quantity at the receiving end diminishes and its price rises.

Relative to whether the Reconvened Plan, the Conference Plan, or the New York Plan should be the one constructed, it would be interesting to learn the estimated value of this power when distributed to those parts of the Eastern United States that would be able to utilize it, and the cost of generating steam power in the same communities. In making such an estimate the class of use would be a major consideration and a conservative load

factor would have to be assumed for the estimate to be fair either to the St. Lawrence power or to the steam power.

The writer understands that steam-electric power can be produced to-day in the principal cities of New York State for a fraction of a cent per kilowatt-hour and that mechanical and electrical engineers admit that further economies are possible; in fact, they state that such economies will come. If steam-electric power can be put on the low-tension bars for 3 or 4 mills per kw-hr, it may be difficult for St. Lawrence power to compete with it over any long-distance transmission and with a normal load factor.

W. S. LEE,[17] M. AM. SOC. C. E. (by letter)[17a].—This paper is extremely pertinent and interesting to the American as well as to the Canadian Engineering Fraternity. The subject is very important and should not be criticized in an unfriendly manner by either one side or the other; the entire Engineering Profession should enter into this discussion in a spirit of determining the best possible development that can be made. This development forms part of the boundary between two great friendly nations, and they will always be mutually interested in it and will have friendly interest in one another. American engineers have too many friends in Canada and *vice versa* for this matter to be discussed in any other way except for the best interests of all concerned; therefore, any discussion that would tend to enlighten the situation before the public as well as the Engineering Profession, aiming toward the development of the best possible plan in this project is, of course, very pleasing to every one.

If this project is carefully studied (as it should be) and properly developed it will be a great acomplishment. If it is poorly developed and mistakes are made, such mistakes will stand out always, and the work will be a poor presentation of the ability of American and Canadian engineers. The writer urges the Engineering Profession to join in a most careful study of the best laid plans, in order to decide on the most economical and enduring construction, so that it may be not only a great advantage to the two countries, but the embodiment of good engineering.

FRANK E. BONNER,[18] M. AM. SOC. C. E. (by letter)[18a].—Proposals for the utilization of the St. Lawrence River present several complex questions. None is more important to the people of the United States than that relating to the merit of the single-stage plan as compared with the two-stage plan for improvement of the International Section of the river for the combined purposes of navigation and power production. Professor Mead's paper reviews this issue with admirable sagacity and candor.

The subject is of timely interest and searching examination by the Engineering Profession constitutes a high public duty. Several years ago, while dealing with various plans presented to the Federal Power Commission for the development of the St. Lawrence, the writer became keenly impressed

[17] Pres., W. S. Lee Eng. Corp., Cons. Engrs., Charlotte, N. C.
[17a] Received by the Secretary September 5, 1933.
[18] Cons. Engr., Piedmont, Calif.
[18a] Received by the Secretary September 22, 1933.

with the probable danger of wasteful and extravagant improvement should the program be guided by professional politicians rather than by competent engineers. It is true, of course, that some other elements must be considered, but, primarily, the entire problem is one of engineering in its broadest aspects.

If the St. Lawrence project is to be launched in the near future by joint effort of the two nations, it is vital that the partnership be based on complete and sympathetic recognition of the rights and problems of each. In order to reach equitable agreement, concessions should be expected on both sides, but in so far as humanly possible these concessions must be evenly balanced or the proposed project will fail for want of public support in one country or the other.

Preliminary negotiations of the past decade (1923–33) have developed that the chief item of disagreement concerns the single-stage plan advocated by representatives of the United States *versus* the two-stage plan preferred by the Canadians. The precipitate acceptance of the Canadian contention in 1932 without adequate explanation naturally provokes a question as to whether the surrender might have been influenced by some temporary political expediency, or whether compensating advantages have been granted the United States in other respects. A satisfactory showing that the proposed treaty does not impose inequitable burdens on the United States should be a necessary prerequisite to unconditional ratification by the Senate.

Professor Mead's analysis clearly demonstrates that the single-stage development is far more favorable than the other from the standpoint of the United States. The original cost will be about $40 000 000 less than the cost of the two-stage plan while at the same time power production is increased, operation costs are reduced, and navigation is served fully as well. The modified single-stage plan proposed by the State of New York offers promise of even larger advantages, but because of uncertainty as to whether the estimates are strictly comparable, definite appraisal cannot be made with assurance. However, in view of the high standing of the engineers responsible for the New York plan, it may safely be assumed that the single-stage plan considered by the Joint Board can be further improved with consequent widening of the margin of advantage over the two-stage plan.

In the face of these established facts it is difficult to understand the Canadian preference for the two-stage plan. The reasons advanced seem rather vague and indefinite. Apparently, they relate mainly to local prejudice against marginal flowage and fear by some interests of aggravating ice conditions along the lower river. Although sentiment favoring the two-stage plan may be of sufficient weight to justify preference for the more costly development by Canada, it is not evident that the actuating factors are of such a character as to warrant the United States assuming a greatly increased burden.

If early development of the St. Lawrence is more urgently needed by the economic requirements of the United States it might be desirable to undertake more than a proportionate share of the obligations in order to expedite completion. Apparently, however, such is not the case. Available informa-

tion shows no pressing need for proceeding with the project at present in so far as the United States is concerned. Doubtless, it will be a desirable improvement eventually, but under present conditions the enterprise is more likely to be a liability than an asset.

The project will provide only two things—a new transportation facility and a huge block of electric energy. These will be useful only to the extent that they may either supply requirements which cannot be met by present equipment or furnish service at a sufficiently lower cost to displace existing competitive facilities to the general public benefit. Active agitation for the St. Lawrence Waterway began during the post-war period, when existing transportation systems were heavily overloaded; but the situation has completely changed and there is now a great surplus of transportation. The same is true of electric power. Under the circumstances, the St. Lawrence project will have to meet the test of competition, and construction will be justified only when it can serve much more economically than the facilities it will displace.

The navigation project involves considerably more of course than the items included in Professor Mead's summary, which is confined to the International Rapids Section. The many ramifications of the entire navigation enterprise are too complex for more than incidental attention in connection with the present discussion. It should be observed, however, that an outlay of several hundred million dollars will be required of the United States, exclusive of extensive harbor improvements, and comprehensive analysis[19] shows that if tolls are placed on the traffic, as they ought to be, sufficient to cover the interest, maintenance, and operation charges, the venture offers little hope of reducing present charges for transportation between Lake ports and the Atlantic seaboard.

It is realized that Professor Mead's analysis relating to cost and net income of power production was intended merely for the purpose of comparing the relative values of the different development plans, but it is possible that the method of presentation may convey some misapprehension regarding the margin of profit possible from this feature of the project and may thus lead to the conclusion that the United States can afford to deal generously with Canada.

The available output will be subject to considerable seasonal variation. Due to the combined effect of low water and reduced head during the winter months, the production will be lowest at the time of highest demand in the electric market. Under the circumstances the continuous primary power will be represented by the winter capability of the generating plant.

Under natural conditions the winter discharge from Lake Ontario occasionally diminishes to about 175 000 sec-ft but, with the moderate regulation planned, the flow will seldom fall below 190 000 sec-ft. Discharge of 200 000 sec-ft may be maintained approximately 90% of the time and for 64% of the time 210 000 sec-ft would be available (50-year period, 1881–1930).[20] For

[19] "The St. Lawrence Navigation and Power Project," pub. by the Brookings Inst., 1929.
[20] Plate 19, of Appendix B (revised to November, 1930), Joint Board Rept. of 1926.

this same period a flow of 235 000 sec-ft would be available about 48% of the time. In some years this would extend through 10 months of the year, but during dry cycles the discharge may be considerably less for as long as 33 consecutive months. During the period, 1923–1927, the controlled flow would exceed 225 000 sec-ft in only 11 of the 60 months. In view of the irregularity of the higher flows it is clear that part of the potential power is of very poor quality and of doubtful usefulness.

Taking 76.4 ft as the average winter head (Barnhart Island Plan) and the discharge at 190 000 sec-ft, the dependable primary power will amount to 1 402 158 hp. The secondary power of good quality will be represented by that excess which may be produced from discharges up to 210 000 sec-ft. This block may be estimated as follows. Let T_1, T_2, and T_3 represent the proportional time elements, respectively, of (*a*) summer power (9 months), from discharges of 200 000 to 210 000 sec-ft; (*b*) the winter power (3 months), from discharges of 190 000 to 200 000 sec-ft; and (*c*) the winter power from discharges of 200 000 to 210 000 sec-ft. Then, from the foregoing time-flow percentages: $T_1 = 0.75 - \frac{(0.75 - 0.64)\,\frac{0.75 - 0.64}{0.90 - 0.64}}{2} = 0.72673$; $T_2 = 0.25\,\frac{1.00 + 0.90}{2} = 0.2375$; and $T_3 = \frac{(0.90 - 0.75)\,\frac{0.90 - 0.75}{0.90 - 0.64}}{2} = 0.04327$.

The good quality secondary power from discharges up to 210 000 sec-ft equals [86.1 (200 000 × 0.75 + 10 000 T_1) + 76.4 (190 000 × 0.25 + 10 000 T_2 + 10 000 T_3)] × 0.096594 − 1 402 158 = (13 540 714.73 + 3 843 508.28) 0.096594 − 1 402 158 = 277 054 hp (average). Similarly, the poor quality secondary output produced from discharges ranging from 210 000 to 235 000 sec-ft may be computed, as follows: 25 000 × 86.1 × $\frac{(0.48 + 0.64)}{2}$ × 0.096594 = 116 429 hp (average).

The estimated average annual production is summarized in Table 7.

TABLE 7.—AVERAGE ANNUAL POWER PRODUCTION.

Class	Horse-power	Kilowatts	Kilowatt-hours
Primary	1 402 158	1 046 010	9 163 000 000
Secondary-good quality	277 054	206 682	1 810 500 000
Secondary-poor quality	116 429	86 856	760 900 000
Total	1 795 641	1 339 548	11 734 400 000
United States' share (50%)	.....	.....	5 867 200 000

The construction cost of the works chargeable to power, as adjusted by Professor Mead for an installation of 2 200 000 hp at Barnhart Island, amounts to $146 645 500 (Item No. 59, Table 1). This is based on the assumption that the cost of the dams, dikes, channel excavation, etc., useful and necessary for both navigation and power, will be apportioned equally, 50% to

each. Power may be called upon to bear a heavier share of the burden, but for present purposes the more optimistic proration will be adopted. Interest during construction amounting to 12½% (5 years at 5% × ½), or $18 330 700, will bring the total outlay to $164 976 200 or, say, $165 000 000, by the time the plant is ready for operation. The investment of the United States in the construction, therefore, may be taken as $82 500 000.

The annual cost of the power development facilities of the United States estimated on a commercial basis for comparison with competitive sources is, as follows:

Interest ($82 500 000 @ 5%)	$4 125 000
Amortization (40 yr @ 5%; factor, 0.0083)	684 750
Depreciation ($50 000 000, 33 yr; factor, 0.0125)	625 000
Operation and maintenance (60 cents per hp installed)	660 000
General expenses, legal and management (15 cents per hp installed)	165 000
Federal license fee (90% capacity 669 000 hp @ 25 cents)	167 400
New York State charges @ 1.5%	1 237 500
Total	$7 664 650

The foregoing financial set-up assumes the development undertaken by some form of a public corporation probably financed by revenue bonds and with obligation to pay the customary Federal charges and State fees equivalent to taxes.

An important additional item of cost in a large development of this kind arises from the idle investment during the years immediately succeeding completion while the market is being built up to the full capacity of the project. Contracts for the Boulder Canyon power were negotiated on a basis of a 4-year absorption period, but it is now evident that a considerably longer time will be required for the growth and adjustments of the tributary markets to utilize the available output. In the report of the New York State Commission eight years was assumed as the absorption period[21] for the St. Lawrence power. On account of possible savings by means of progressive partial installation it seems reasonable to assume five years and estimate the added capital charge to the project by taking one-half the annual costs for such period. This amounts to $19 161 625 and increases the total investment at the time the plant reaches full operation to $101 662 000. Interest and amortization on the added capital charges increase the annual costs to a total of $8 781 800.

One-half the primary output that will be available to the United States amounts to 4 581 500 000 kw-hr. This presumes discontinuing the diversion of approximately 20 000 sec-ft which supplies the existing Massena Plant and instead serving equivalent energy amounting to about 80 000 hp of peak and

[21] Rept. of the St. Lawrence Power Development Comm. (State of New York), January 15, 1931, p. 59.

400 000 000 kw-hr per year from the main plant. The primary output available for disposal is thus decreased to 4 181 500 000 kw-hr. At the same time, by crediting proportionate operation, maintenance, and other appropriate costs amounting to $207 800, the annual cost of the generating project is reduced to $8 574 000.

Forecast of the probable revenue from the sale of power opens up a complicated subject which can be treated here in broad outline only. Obviously, the secondary power, if utilized at all, will have to be sold at very low rates for seasonal industrial purposes in the immediate locality of the plant. Under present conditions, such demand would be negligible, but eventually substantial fractions of these irregular energy blocks might be salvaged by new industries attracted by low-cost power. The share of the United States of the secondary output of good quality averages 905 250 000 kw-hr, and of the poor quality blocks, 380 450 000 kw-hr. Disposal of 70% of the former at 1.0 mill and 50% of the latter at 0.5 mill would probably represent about the maximum expectation. The gross return on such basis would average about $725 000 annually.

The primary power production will have to be marketed at high load factor by wholesaling the large industries induced to migrate to the river, or by transmission to the distant metropolitan load centers. The Boston, Mass., and the Newark, N. J., Districts probably offer the most promising prospects. Transmission in either case would be about 325 miles. New York City could be reached with about the same length of line, but in competition with local steam-generated energy the value of the imported power would be considerably less due to the costly underground cable facilities required for access to the city sub-stations. Present dimensions and characteristics of the load of the Boston and Newark Districts might cause some question about the capacity of these markets to utilize the St. Lawrence primary energy as base load at high load factor about ten years hence. It is recognized that the reputations of most authoritative prognosticators relative to the probable growth of electric load have suffered heavily during the past few years, but nevertheless the assumption is made that in due course the Boston and Newark Districts, with taps serving some of the secondary load centers en route, could each absorb about one-half the St. Lawrence primary energy at 85% utilization factor and with minimum load factor of 80 per cent. This combination imposes peaks about 6% in excess of the average primary output, but it is considered that the resulting fluctuation of discharge would be within the permissible limits during the non-navigation season and, under the more severe requirements for constant flow during the summer, the variation of output could be accomplished without seriously affecting the characteristics or value of the secondary power. Without going into details, it may be assumed that the cost of generation by local steam plants at 80% load factor will be approximately 4½ mills per kw-hr as developed by the report of the New York State Commission.[22] This rate, therefore, represents the maximum market value of the imported power delivered at load centers.

[22] Rept. of St. Lawrence Power Development Comm. (State of New York), January 15, 1931, pp. 80-82.

For transmission of the primary output of 4 181 500 000 kw-hr (477 340 average kw) with a utilization factor of 85%, load factor of 80%, and transmission efficiency of 87% (line, 9.5%; step-down, 1%; and synchronous condensers, 2.5%), the necessary peak capacity at delivery terminals will total 441 241 kw. This will require four 220-kv circuits (two each to Boston and Newark), with delivery capacity of 110 000 kw each. At $25 000 per circuit-mile, including right of way, the lines will cost $32 500 000. Terminal sub-stations with synchronous condensers at $16 per kw and sectionalizing and switching stations will cost $7 500 000 more, thus making the total investment in transmission facilities $40 000 000. The annual cost on an 9% basis (interest, 5%; amortization, 1%; and depreciation, operation, and maintenance, 3%) will amount to $3 600 000, not including taxes which, if estimated at 1.5% of the investment, would increase the total annual cost to $4 200 000.

Steam stand-by, equivalent to 50% of the delivery capacity, or 220 000 kw, will be necessary to assure reliability of service comparable with local production. If estimated at $80 per kw the investment will be $17 600 000 and the annual cost, exclusive of taxes, $2 436 500 (interest, 5%; amortization, 1%; depreciation, 3%; operation and maintenance, $2.25 per kw; general expense, 50 cents per kw; and, stand-by fuel, 0.25 ton @ $4.50 per kw). Taxes at 1.5% will increase the annual stand-by cost to $2 700 500.

The cost of the delivered energy amounting to 3 092 219 250 kw-hr (4 181 500 000 × 85% utilization factor × 87% transmission efficiency) is summarized by the following tabulation:

	Including taxes	Without taxes on transmission facilities
Production cost ($8 574 000 less $725 000 return from secondary)	$7 849 000	$7 849 000
Transmission and sub-station cost	4 200 000	3 600 000
Stand-by cost	2 700 500	2 436 500
Total delivery cost	$14 749 500	$13 885 500
Cost per kilowatt-hour delivered, in mills	4.77	4.49

It is apparent from these general figures that in competition with equivalent local steam-plant generation, at approximately 4½ mills, the introduction of St. Lawrence power into the large metropolitan districts fails to provide a very favorable possibility even though the transmission lines, sub-stations, and stand-by plants should be relieved of taxes. In any event, it will be clear that there is no hope of obtaining any large profits or extensive public benefits from the enterprise.

The alternative of endeavoring to market the output in the immediate vicinity of the generating plant by building up large electro-chemical and electro-metallurgical works, although largely speculative, seems to offer a somewhat beter chance of accomplishment. The development of such a project, however, will require time, a vast expenditure of money, and the

closing of industries in other sections. The feasibility will depend largely on the cheapness of the power rates. In order to attract large industrial load, it is doubtful whether the average rate can be higher than $2\frac{1}{4}$ mills per kw-hr ($14.70 per average hp). Disposal of the primary output at such rate, with 85% utilization factor, would produce revenue of $7 997 119. This would cover the generating cost by only a slender margin after taking credit for the possible return of $725 000 from secondary power.

From the foregoing, it will be evident that the situation offers no assurance of power revenue in excess of that needed to meet the bare carrying charges. Substantial increase in the capital outlay as involved with the adoption of the two-stage plan can easily thrust the project from the border line of financial feasibility into the outer zone of economic impractibility. Hence, the importance of this question of development plan to the United States. In Canada, the situation is different on account of large load centers being close to the power site. The extra cost of about $40 000 000 for the two-stage development is chargeable mostly to the power facilities and would raise production costs approximately 25 per cent. With such an increase, the marketability of the output would be extremely doubtful. Under the circumstances, it would appear that if the two-stage plan is adopted, Canada, in all fairness, should be willing to assume the additional cost.

RUFUS W. PUTNAM,[23] M. AM. SOC. C. E. (by letter).[23a]—Reference is made in this paper to a few of the navigation aspects of the power developments under consideration; furthermore, in the tables showing comparative costs, items appear for costs of navigation features embodied in the three plans. This discussion proposes to set forth the writer's views on the relative merits of these plans for water-power development as affecting the navigation interests involved.

Factors affecting the utilization of the waterway by vessels include channel dimensions and curvature, cross-currents, ice conditions, and lockages. Table 8 sets forth some of these factors.

TABLE 8.—FACTORS AFFECTING UTILIZATION OF THE WATERWAY BY VESSELS

	Plan *A* (Crysler)	Plan *B* (Barnhart)	Plan *C* (New York)
Length of canal, in miles	9	8	0
Number of locks	3	2	2*

* In flight.

Channels.—As to channel widths, depths, and curvature, the same specifications have been followed in each plan and, therefore, no differences exist. Plan *A* and Plan *B* are about equal as to length of restricted channel, but they have a disadvantage compared with Plan *C*. A ship will require about an hour more to navigate the channels available with either the Crysler two-stage project, or the Barnhart single-stage project, than it will with the New York single-stage project. Assuming the ultimate capacity provided

[23] Director, Chicago Regional Port Comm., Chicago, Ill.

[23a] Received by the Secretary October 2, 1933.

for in Plans *A* and *B*, as 24 000 000 tons per year, with average cargoes of 5 000 tons per vessel, the vessel-hours lost would be 4 800. At a cost of $30 per hr the annual loss becomes $144 000, which, capitalized at 5%, represents a capital advantage in favor of Plan *C* of $7 200 000. This represents a minimum loss based on capacity tonnage for Plans *A* or *B*, for if smaller average cargoes are assumed the number of vessel-hours lost increases proportionately, but the hourly vessel cost does not decrease in like proportion.

Lockages.—Plan *A* will require three lockages per trip per vessel, while Plans *B* and *C* will require only two. It is customary to allow an average of 1 hr per lockage, even though many passages are made in much less time. The records at the locks at Sault Ste. Marie bear out this conclusion.

The New York Plan (*C*) has two locks in flight, while the locks provided by Plans *A* and *B* are separated sufficiently to have no added influence on time of transit. The Joint Board of Engineers estimates the annual capacity of a single dock development at 24 000 000 tons, but places that of an improvement involving a set of flight locks at 16 000 000 tons per year. The average time of 1 hr per transit through a single lock, therefore, becomes 1½ hr for each lock in a set of flight locks. For comparative purposes, therefore, lockage times are as follows: Plan *A*, 3 hr; Plan *B*, 2 hr; and, Plan *C*, 3 hr.

Using the same assumptions as to size and costs of vessels and annual capacity of the waterway as before, Plan *B* has a capitalized advantage of $7 200 000 as regards Plans *A* and *C*. Combining the capitalized value of lock delays with those involved in channel transit, it appears that Plan *A* entails a loss of $7 200 000 as compared with Plans *B* and *C*, while Plans *B* and *C* are of equal value in these respects.

Capacity.—A further consideration involved is that of the ultimate capacity of the three projects which is 24 000 000 tons per year for Plans *A* and *B*, and 16 000 000 tons per year for projects involving flight locks, according to the report by the Joint Board of Engineers. If Plan *C* is followed, it will be necessary to install a parallel flight of locks in order to obtain the same capacity as that afforded by Plans *A* and *B*. This will involve an expense of between $7 000 000 and $8 000 000. As far as ice conditions and cross-currents are concerned, it does not appear from any of the plans available that any difference exists between Plans *A*, *B*, and *C*.

Summary.—The writer considers that, from the point of view of the interests of navigation alone, Plan *B* is superior to both Plans *A* and *C* which are of about equal ultimate value.

JAMES W. RICKEY,[24] M. AM. SOC. C. E. (by letter).[24a]—Engineers are under a moral obligation to advise the public and to assist in forming an enlightened public opinion on such matters as the development of water power along the International Section of the St. Lawrence River, which they are particularly qualified to discuss.

The author's analysis points unmistakably to the conclusion that the development of this section of the river in a single-stage plan is superior in

[24] Chf. Hydr. Engr., Aluminum Co. of America, Pittsburgh, Pa.

[24a] Received by the Secretary October 6, 1933.

every engineering aspect to its development in a two-stage plan. This raises the question as to why the two-stage plan was incorporated in the Treaty, now (November, 1933) awaiting ratification. A study of the various reports apparently provides the answer, namely, that an agreement between the Canadian and the American Sections of the Joint Board of Engineers must be reached before any development can be undertaken, and that the Canadian Section would agree to the two-stage plan and no other, notwithstanding the less satisfactory navigation facilities, the reduced power output, and the greatly increased cost of the two-stage plan.

It is vitally necessary that the people of both countries and their responsible representatives be fully acquainted with all the facts before adopting a definite plan for the public development of this great National resource. The facts clearly made evident by the paper are that, when using the same unit prices in both estimates of cost, the single-stage Plan *B* will be worth about $94 000 000 more and will cost about $38 000 000 less than the two-stage Plan *A*; and, that when similar comparison is made between Plan *C* and Plan *A* the corresponding sums will be very much greater. (See Table 3, Item No. 9, and Table 1, Item No. 18.)

The author is to be congratulated highly upon his clear, unbiased analysis of the facts set forth in the various published Government reports. The paper and the discussion incident thereto should be of great assistance in securing the early execution of this project and in accordance with the best interests of all concerned.

WALTER M. SMITH,[25] M. AM. SOC. C. E. (by letter)[25a].—The canals at the International Section of the St. Lawrence River, are somewhat analogous to the Sault Ste Marie Canal, although the class of boats passing through it will be quite different. As about 60% of the freight carried through the "Soo" Canal is iron ore, a type of vessel not sufficiently strong for ocean going, has been developed for this traffic. Therefore, conclusions drawn from traffic through the "Soo" Canal can only be applied in a very general manner to the International Section of the St. Lawrence River.

Table 9 shows statistics of traffic through the "Soo" Canal from which some general conclusions may be drawn. It will be seen from Table 9(*a*), Item No. 11, for example, that in 1929 in moderately heavy traffic the speed in the canal was about 2 miles per hr, whereas in Table 9(*b*), Item No. 23, it is shown that in maximum traffic through each lock it was from 0.8 mile to 1.4 miles per hr. Thus, it will be seen that as traffic increases the speed is cut down and all vessels lose more time in transit. The canal at the "Soo" is 300 ft wide against 200 ft proposed for the canal in Professor Mead's study, and yet the engineers in charge claim that it is not sufficiently wide for two-way traffic. It will be seen that the average size of vessel in the "Soo" Canal in 1929 was 6 250 dead weight tons (Table 9(*a*), Item No. 5,

[25] Acting Chf. Engr., State Dept. of Public Works and Bldgs., Div. of Waterways, Chicago, Ill.

[25a] Received by the Secretary October 12, 1933.

Column (6)); therefore, this size of vessel will be used for a comparison in this study.

The Poe Lock at the "Soo" Canal is the same length as the proposed locks on the canal being discussed, but is 100 ft wide instead of 80 ft as is proposed for the locks in question. It will be seen from Table 9(*b*) (Item

TABLE 9.—STATISTICS FOR SAULT STE MARIE CANAL

Item No. (1)	Description (2)	Names of Locks: Poe (3)	Davis (4)	Sabin (5)	Totals (6)
	(*a*) 1929 STATISTICS				
1	Number of lockages	3 852	3 910	3 790	11 553
2	Number of vessels	4 856	5 632	5 721	16 209
3	Total tonnage	14 688 835	36 037 355	50 590 508	101 316 698
4	Average tonnage per lockage	3 800	9 200	13 300	8 800
5	Average tonnage per vessel	3 000	6 400	8 800	6 250
6	Average number of lockages per day	10.5	10.7	10.0	31.7
7	Average time per lockage, in seconds	1 370	1 996	2 244	.....
8	Lift, in feet (approximately)	21	21	21	.....
9	Average time in canal, in minutes	.....	.....	.....	70
10	Length of canal, including lockage, in miles	.....	.....	.....	1.75
11	Speed in canal, in miles per hour	1.94	1.96	2.26	.....
	(*b*) STATISTICS FOR MAXIMUM YEAR AT EACH LOCK				
12	Year	1906	1918	1923	.....
13	Number of lockages	5 845	4 676	3 691	.....
14	Number of vessels	9 968	7 259	6 675	.....
15	Vessel interval, in minutes	52	72	80	.....
16	Total tonnage	43 083 490	60 946 348	55 591 159	.....
17	Average tonnage per lockage	7 400	13 000	15 000	.....
18	Average tonnage per vessel	4 300	8 400	8 400	.....
19	Average lockage per day	16	13	10	.....
20	Average time per lockage, in seconds	2 071	2 268	2 754	.....
21	Average time through the canal, including lockage, in minutes	153	91	111	.....
22	Speed in canal, in miles per hour	0.8	1.40	1.10	.....
23	Average Tonnage per Year, for:				
24	20 years	25 854 000	.....	.....	.....
25	18 years	27 480 000	34 600 000	.....	.....
26	16 years	31 420 000	38 437 000	45 070 000	.....
27	5 years	35 592 000	56 635 000	49 467 000	.....
28	1 year	43 083 000	60 946 000	55 591 000	.....

No. 12, Column (3)) that 43 083 490 tons of freight passed through the Poe Lock in 1906, but it will be noticed that there was an average of 1.7 vessels per lockage, and making a correction for an average of only a single vessel of average size per lockage would give a traffic of about 26 000 000 tons per year, which is double the estimate of the Joint Board of Engineers for the capacity through the locks in question, as long as they are single locks. It will also be seen that if there had been open-water approaches to the Poe Lock the capacity would be much higher because the average time per lockage was only 34½ min, which would give 42 lockages per day mathematically possible, whereas the number was only 16, or about 40% of capacity. It will also be seen that it took 2 hr to navigate the 1¾ miles of canal, or three and one-half times as long as to go through the lock.

The data for the Davis and Sabin Locks are not comparable with the proposed locks as they are 1 300 ft long, although they are the same width (80 ft) as the proposed locks. On account of the great increase in traffic in the "Soo" Canal these locks were built to accommodate two of the largest

lake steamers at the same time. It is interesting, however, as showing what capacity may be reached in locking. No other canal in existence has ever passed more than 60% of the traffic of one of these locks.

It will be noted that the great traffic through the Poe Lock was in 1906 long before either of the other two locks were built, the Davis Lock having been put in commission in 1914 and the Sabin Lock in 1919. After the completion of these two locks the passenger traffic was turned over to the Poe Lock and the two large locks turned over to freight traffic almost exclusively. Therefore, the 14 684 835, shown for the Poe Lock in 1929, is registered tonnage; in all other cases, it is dead weight tonnage.

It will be noted that as the tonnage fell from 43 000 000 to 14 700 000 the locking time decreased from 34.5 to 22.8 min, and the speed in the canal increased from 0.8 mile to 1.94 miles per hr. This shows vividly the bottle-neck effect of any narrow canal in slowing down traffic as the volume of traffic increases.

Assume that passing places will be made in the proposed canal sufficiently close together for a vessel to pass from one to the other in the same time as it takes to go through the lock. In this case the mathematically possible maximum traffic would be 1 vessel every 25 min through the lock; but every vessel will be waiting one-half the time throughout its passage through the canal.

Assume next the existence of an ideal condition of a series of standard-sized vessels, 6 250 tons, moving with clock-like precision, one going through the lock every 25 min. This would give 58 lockages per day, or a capacity of $58 \times 6\ 250 = 362\ 500$ tons per day, or 132 000 000 tons per year. Assume also that it is possible to attain 33% of this tonnage as was done at the Poe Lock in 1906. It will then be seen that it is not the lock that is the bottle-neck, but the canal. At the Poe Lock with a canal 300 ft wide, only an average speed of 0.8 mile per hr was attained in the canal; therefore, assume a speed of 1 mile per hr in the canal. The upper canal at Crysler Island is 2.5 miles long and the lower one at Barnhart Island, 7.5 miles long; a total length of 10 miles. Therefore, it will take a vessel 11 hr, including the time through lockages, to get through the canal.

In the Massena Point scheme there is a proposed channel, 450 ft wide, leading up to the lock down stream for 2 miles, and open lake navigation on the up-stream side so that vessels can come and go at open-river navigation speed except for the 2 miles, where the speed will have to be reduced somewhat, although vessels can pass at moderate speed. To pass through the flight of two locks having a lift equal to the three locks in the other scheme will require about 1 hr. This would give 24 lockages per day, or $24 \times 6\ 250 = 150\ 000$ tons per day, or 54 750 000 tons per year. As there is no bottle-neck canal here, these locks can certainly pass more than the canals and locks in the Crysler Island Plan. The vessels approaching these locks could certainly run 10 miles per hr in the open river and at least 8 miles per hr in the 450-ft channel; therefore, it will require only 2 hr to go 10 miles, including the lockage. This will be a saving in time of 9 hr.

It costs about $64 per hr to operate a vessel of 6 250 tons; thus, the extra expense for each vessel in the Crysler Island Plan will be $64 × 9 = $576 and 19 vessels will be $576 × 19 = $10 944 per day, or $10 944 × 365 = $3 994 560 per year. This amount capitalized at 5% gives $79 891 200. Therefore, under the foregoing assumptions, other things being equal, an additional expenditure of about $80 000 000 would be justified on the Massena Point Plan over and above what the Crysler Island Plan would cost for navigation alone.

In studying the advisability of building a sea-level or lock canal in 1906, the Board of Consulting Engineers for the Panama Canal estimated that the time it would require a vessel of the size used herein would be from 8.6 hr for 10 vessels per day, to 11.1 hr for 30 vessels per day, to pass through the sea-level canal, with passing places 2.5 miles apart. The same size of vessel could pass through the 85-ft level lock canal, including the six locks, in 9.5 hr for 10 vessels, and 10.0 hr for 30 vessels, per day. This was a speed of from 7.7 to 7.1 miles per hr in the lock canal, allowing 3 hr for lockage, and an average speed of from 5.7 to 4.5 miles for 10 vessels per day per hour, with an average of 30 vessels per day, for the sea-level canal with passing places 2.5 miles apart. The dimensions assumed for the two canals are given in Table 10.

TABLE 10.—Dimensions of Sea-Level and Lock Canals Proposed for Panama Canal

Item No. (1)	Bottom width, in feet (2)	Length of sea-level canal, in miles (3)	Length of 85-ft level canal, in miles (4)
1	150	21.1	....
2	200	18.8	4.7
3	300	3.9	7.7
4	340	0.3	....
5	350	0.5	....
6	500	....	8.2
7	800	....	3.8
8	1 000	....	19.5
9	Unlimited	4.7	5.8
10	Total....	49.3	49.7

It will be noticed that even with only 12.4 of the 49.7 miles of the lock canal, with a bottom width of less than 500 ft, the speed of vessels with 10 vessels per day, including lockages, was 7.7 miles per hr, and for 30 vessels, 7.1 miles per hr, in the canal.

The high speed of vessels assumed for the lock canal at Panama is explained by the fact that 75% of the canal is 500 ft, or more, and 50%, 1 000 ft, or more, wide. Therefore, 50% of the distance is practically open-lake navigation. In channels 500 ft wide, vessels of the size herein assumed can pass each other at full speed. With channels 300 ft wide they can pass at reduced speed without stopping; but at 200 ft, or less, in width one vessel will have to stop and the other greatly reduce its speed.

Even if it is assumed that the vessels passing through the canals in the Crysler Island Plan can average 2 miles per hr, including time lost at meet-

ing places, it will take each vessel about 6.5 hr to pass through the 10 miles of canal and locks, or 4.5 hr more than in the Massena Point Plan. This is as favorable an assumption as can be made and even under these conditions the time lost will be valued at about $2 000 000, and capitalized at 5%, about $40 000 000.

When the traffic is increased sufficiently to require an enlargement of the canals and locks the capacity of the Massena Point Project can be doubled approximately by building an additional flight of locks alongside the other at a cost of about 70% of the cost of the original flight of locks and appurtenances. As the cost of the original flight of locks and other work solely for navigation is estimated at $21 138 000, this increase will be $21 138 000 × 70% = $14 796 600. To increase the capacity of the Crysler Island Project in the same proportion will require an increase of probably nearer 65%, and as the estimate of cost of work purely for navigation is $34 138 000, the increase in cost would be $34 138 000 × 65% = $22 189 700, or an increase of $7 393 100.

It is evident from the foregoing analysis that, other things being equal, the Massena Point Plan is far preferable to the Crysler Island Project from a standpoint of economy. In the Barnhart Island Project there is a moderate gain over the Crysler Island Project, but as the length is about the same as that of the lower canal in the Crysler Island Project, the gain would be only about 25% as great as for the Massena Point Project.

J. W. BEARDSLEY,[26] M. AM. SOC. C. E. (by letter)[26a].—The author has presented a critical study of the three St. Lawrence River projects submitted to him for comparison, and has arranged his finding in exceptionally convenient form for ready reference. He states that his knowledge of the rapids of the river is limited. He implies that there are phases of the problem other than the engineering that should be considered. He also implies a compliment to the Canadian Government on account of its policy of re-appointing the members of its Commission, and, possibly, a criticism of the United States in not following a similar policy. If these implications had been clearly stated, the writer would have agreed heartily.

The report of the Deep Waterways Commission (1900)[27] relates specifically to navigation problems and estimates of cost for channels, 21 and 30 ft deep. In that report, the writer discusses the geology of this section.[28] Broadly, the rapid St. Lawrence River flows along a ridge and the sluggish Grass River in a valley, 50 ft lower and 3 miles distant.

In one case while inspecting wash drill borings, the writer noted that the overlying boulder clay was being penetrated with difficulty. The drill broke through it, and the following 70 ft in blue clay was drilled in about 30 min, or as rapidly as connections could be made. Similar formation was found at Massena Point and along the southeast shore of Cornwall Island. It was this formation that caused the failure of one of the piers during the construction

[26] Cons. Engr., Syracuse, N. Y.
[26a] Received by the Secretary October 19, 1933.
[27] H. R. Doc. No. 149, 56th Congress, 2d Session.
[28] *Loc cit.*, Appendix 11, p. 398.

of the Ontario and Western Railway Bridge. The foundations were on a thin layer of boulder clay through which the increased velocity of the stream cut down to the clay formation. Boulder clay, found previously when making sub-surface explorations, had been discovered resting on rock in place. To find it resting on a considerable body of blue clay was a new experience to the writer—a condition that may be explained if secondary clay deposits from a prior glacier had been bridged by the ice-cap of a subsequent glacier.

In another case, the formation of frazil in the forebay was creating an anchor-ice barrier in the tail-race, which reduced the head by one-half. During past years the northern water-power companies have had considerable trouble in maintaining a free tail-race under such conditions—an important item when continuous electrical service is financially necessary.

These facts have been stated to indicate that at least several years of critical observation of the St. Lawrence is needed to estimate properly the effects of ice-jams, anchor ice, frazil, water-stages, etc., and to counteract them. In general, the northeastern part of the United States is well provided with electrical energy. New electrical devices for household use are appearing almost daily, and the various systems are constantly soliciting consumers for surplus power.

A circle described about Massena Point as a center, with a radius of about 300 miles, will include Boston, Mass., New York, N. Y., and Buffalo, N. Y. The principal systems with their subsidiaries operating in New York State are the Niagara Hudson, the Associated Gas and Electric, and the New York Edison. Of these three systems, the Niagara Hudson serves the greater area (and probably the greater population, excluding New York City) of the State and more definitely the northern half, including the area adjacent to the St. Lawrence River. It is interconnected with systems serving contiguous territory. A few facts regarding that system will illustrate the points the writer wishes to make.

More than 60% of the total capacity of the Niagara Hudson Company, (1 695 000 kw), is in water-power plants. The load factor on the day of maximum demand in 1932 was 78 per cent. The number of farms estimated within the franchise area is 74 374 (the total in the State was 159 806 in 1930); the number of farm consumers is 27 184, or 38% of the farms in the franchise area. The annual report to stockholders (1932) shows an average farm rate, in cents per kilowatt-hour, as follows: 1929 = 7.384; 1930 = 6.764; 1931 = 6.229; and 1932 = 6.059. (The average rate for residence customers was similarly reduced from 5.028 cents per kw-hr in 1929 to 4.658 cents in 1932). Due to unfavorable location, or greater distance from existing transmission and distribution lines, the rate for many of the remaining farms wishing electrical service would be considerably higher even if no charge were made for the energy supplied.

In 1932, the number of shareholders of common stock was 98 300, and dividends were paid to the amount of $10 471 878, but reductions have been made in the 1933 dividends. It seems reasonable to assume that many consumers of all classes are holders of common stock, and that any sharp compe-

tition or undue volume of surplus energy will cause a considerable reduction in dividends, offsetting any practical reduction in rates. It is generally conceded that even if power was furnished free, the reduction in rates in many cases would be nominal because of the total costs of distribution.

The proposed development of the St. Lawrence River in the near future would throw into a field, already amply supplied, nearly 2 000 000 hp of energy. Can practical use be made of it without the serious depreciation of existing plants? A complete plan should show how this block of power will be marketed at a benefit to the entire State and community.

The sum of the cost of Plans *A*, *B*, and *C* given in Table 1, Item No. 35 (or Item No. 85), divided by the sum of primary and secondary horse-power, Table 1, Item No. 9 (or Item No. 59), gives an average development of about $80. In view of the continuing improvements in steam power with its advantages of location favorable for transportation, markets, labor, and fuel, this average cost is not financially attractive.

On the St. Lawrence River the development of power and of navigation are closely related and interdependent. When navigation is possible for vessels of about 25-ft draft, the benefits to Canada and to several of the Northern Central States will be large. The effect on the Barge Canal and the industries of New York State and New England will doubtless be harmful. In any case they will require broad re-adjustment.

There is no doubt about the ultimate development of the St. Lawrence. However, present conditions throughout the nation, with large electrical projects under way on the Colorado River, the Tennessee River, and in the Columbia River Basin, suggest at least that the St. Lawrence development can be postponed advantageously to both countries. Reduced utility rates is a popular slogan—a vote getter. Electrical appliances are no longer luxuries in American homes. If the excessive rates and unfair practices charged[29] against some companies are true, they can be regulated by equitable laws far more economically for the average citizen than by throwing a huge block of power, without definite plans for its utilization, into an area now provided with surplus power. The writer has made no study of conditions in Canada. If they are similar to those in the United States, the safest plan would be to delay construction until the profitable use of the additional power arises.

It is an enticing vision to plan great power developments, interconnected throughout the nation, technically and financially operated at reasonable profits, furnishing energy to consumers at fair rates, reducing drudgery, and making more profitable the increasing hours of leisure. Such a plan seems to be the goal of at least a few of existing operating and holding companies. Some have exploited the public until faith is lost in all. Will plants operated by the Government reach the desired goal? Does the history of Muscle Shoals prove it? The possibilities of the St. Lawrence project will remain "available assets without depreciation" until utilized.

Conclusion.—The writer's conclusion is a repetition of the last sentence of Professor Mead's "Synopsis," with a modification of his final words, namely,

[29] *Atlantic Monthly*, July, 1933.

it will be a serious mistake and a misfortune to both the United States and Canada, if the treaty now pending is ratified on the basis of the construction in the near future of works controlling the major hydro-electric power of the St. Lawrence River.

L. F. HARZA,[30] M. AM. SOC. C. E. (by letter).[30a]—The author has done a great service both to Canada and the United States by his critical analysis of the proposed official plans for developing water power along the International Section of the St. Lawrence River. Since he has had no previous connection with any of the St. Lawrence River investigations, his analysis is unbiased by preconceived ideas. On the other hand, the analysis may have fallen into error if the unit prices or other basic assumptions of Plan *C* are comparable with Plans *A* and *B*, a question regarding which a difference of opinion exists.

The excess cost and its equivalent, of the two-project plan over either of the single-project plans, is not so great, even after allowing for possible error, that exceedingly momentous reasons of absolute vital concern to the success of the project would need to be advanced to justify the extra cost and equivalent capitalized operating expense.

It is important to realize that the best navigation facilities, in combination with the best power development, are of paramount importance to Canada and the United States. Although power development is secondary in general to navigation, it is the prime requisite to Ontario and New York. The "best plan" is that which provides, with maximum economy, the most satisfactory combined navigation and power facilities, giving due consideration to: Physical conditions, capital charges, cost of operation, maintenance, renewals and repairs of structures, navigation, amount of primary power developable, reliability and cost of power, etc.

Navigation.—It would seem that navigation merits far greater consideration than it has received in the published reports. Under Plan *A* the ships must pass through about 10 miles of canals and 3 locks as against 8 miles of canals and 2 locks in Plan *B* and only 2 locks—there being no canals—in Plan *C*. For fair comparison, it would be necessary, to add to the cost of Plan *A* the capitalized value of the lost time per year of all vessels expected to navigate this waterway. Competent authority has estimated that 3 hr more, or 6 hr per round trip, is required to navigate the 10 miles of canal and 3 single locks of Plan *A*, as compared with the double lock without canal of Plan *C*. If anticipated traffic in this waterway is to be realized, the capitalized value of this time might well be a deciding factor in favor of Plan *C*.

Arguments Against the Single-Stage Development.—In the last paragraph of his paper, the author states that he can see no valid reasons to justify the two-stage plan which can outweigh the millions of dollars of value in favor of other plans. It would seem proper, therefore, to analyze the arguments advanced against a single-stage development.

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[30] Cons. Engr.; Pres., Harza Eng. Co., Chicago, Ill.

[30a] Received by the Secretary October 16, 1933.

The following objection is stated[31] in the Conference Report:

"If, as an alternative, the International Rapids Section be improved by a single-stage project, then control of flow must be secured by operation of gates and turbines at a power plant which will develop about 2 000 000 horsepower. This is not regarded favorably by the down river interests, since this single plant would serve a large and populous territory both in Canada and the United States, and operation restrictions would be difficult to impose in view of the fact that it would be a dominant interest in the territory served. Again, the additional flooding of land [15 360 acres, of which 9 600 acres are in Canada] and the extent of the embankments, along with some adverse effects at the end of the navigation season, make the single-stage project undesirable."

The "control of flow," as noted in the preceding paragraph, was referred[32] to by a member of the Canadian Section, when testifying before the Canadian Special Committee on the St. Lawrence River development:

"We insist that the control of the flow from Lake Ontario be not connected with a power plant having a capacity of several million horsepower. The ideal plan would be the construction of a control dam for the flow only. But as this plan would involve the loss of a certain quantity of power which may become very valuable, and which has an important value to-day, we agreed to connect this dam with a hydroelectric plant of much less proportions, and which can be operated as a basic plant with a uniform load 24 hours a day. * * * For this reason we favor concentration at two points—'the two-stage development.' The control dam for Lake Ontario would be at Crysler Island, a few miles below the town of Morrisburg."

The safety of the City of Montreal is referred[33] to in a statement in the Treaty:

"The Canadian project of a 2-stage development in the International Section has been adopted instead of the original American project of a single-stage development. The cost is slightly more, but the Canadian officials have felt that the 2-stage development is desirable for many reasons, amongst others for the complete assurance of the safety of the City of Montreal."

In analyzing the foregoing arguments against the single-stage development, it is enlightening to consider carefully the 2 000 000 hp. partly completed (fall of 1932), all-Canadian, single-stage development on the St. Lawrence River near Beauharnois, Que., and much closer to (in fact only about 28 miles above) Montreal. In order to make this development efficient, economical, and in accordance with the latest and best engineering practice, practically every feature of general arrangement was incorporated in it that is so strongly objected to in a single-stage international development, with the result that it stands for the "best plan" from the standpoint of engineering design, of capital costs, of sustaining charges, and of cheapness of product to the Canadian power consumer.

The 83-ft fall between Lake St. Francis and Lake St. Louis is developed at Beauharnois[34] by a single-stage project, where the "control of flow must

[31] See Conference Report, 1929, p. 4, paragraph 9.

[32] See First Annual Rept. of the Power Authority of the State of New York, p. 23.

[33] "Great Lakes-St. Lawrence Deep Waterway Treaty," *Publication No. 347*, Dept. of State, 1932, p. 11.

[34] "Beauharnois Development of the St. Lawrence River," by W. S. Lee, M. Am. Soc. C. E., *Electrical Engineering*, June, 1933, p. 377.

be secured by the operation of gates and turbines at a power plant that will develop about 2 000 000 hp." and hence "a dominant interest in the territory served." The power-house is at the end of a canal, 15 miles long, embanked by dikes on both sides throughout its entire length. The entire normal flow of the river will be used to carry the ultimate load and, when part load is carried, the surplus water will be passed through the sluice-gates yet to be built across the main river channel at a distance of more than 15 miles from the power-house.

Since the general physical conditions obtaining at Beauharnois are substantially the same as those obtaining in the International Section of the river, (except in respect to loss of land by submergence), it would seem that the Beauharnois development stands in mute refutation of the foregoing arguments against a single-stage development under Plan *B* or Plan *C*.

Adoption of the Two-Stage Plan.—The Reconvened Report forms a part of the proposed Treaty between the United States and Canada, now (November, 1933) awaiting ratification. Upon this Treaty depends the joint expenditure of $543 000 000,[35] of which sum, $274 742 000 is allocated to the two-stage development, termed Plan *A* in the paper.

For many years prior to the issuance of the Joint Board's Reconvened Report, dated April 9, 1932, it was known that a wide difference of opinion existed between the Canadian and American Sections as to the relative merits of the two-stage and single-stage plans of development. It was also known that most members of the several American Sections of the Joint Board were strongly opposed to the two-stage plan and that many arguments advanced in support of such plan were not considered by those Sections to be of sufficient importance to justify the excess cost from a standpoint of sound engineering and economic reasoning rather than sentiment.

It seems evident that the two-stage plan was adopted for the simple and single reason that a one-stage plan could not be agreed upon under the Board's instructions to report on "a mutually satisfactory plan for the improvement of the International Rapids Section."[36] The author's analysis would indicate that it represents an unfortunate international compromise, perhaps far from being the best and most economical engineering solution of the problem.

Suggested Procedure.—The great difference of opinion revealed by the author as to the merit of the two-stage *versus* the single-stage plans, raises considerable doubt as to the wisdom of the ratification of the Treaty as it now reads. It would seem that a speedy and satisfactory determination of the "best plan" will be possible only after a final, critical analysis is made of all available data pertinent to Plans *A*, *B*, and *C*, by an International Board, comprising all new members without preconceived or previously advocated or supported ideas to defend. Before such a board the members of previous boards should appear as witnesses, not as principals.

[35] "Great Lakes-St. Lawrence Deep Waterway Treaty," *Publication No. 347*, Dept. of State, 1932, p. 10.

[36] See Rept. of Joint Board of Engineers (Reconvened), p. 3, paragraph 6.

Unquestionably, either country has the right to prefer and to insist upon the adoption of a plan which is more expensive than the "best plan;" but in doing so, the right of the taxpayer and the ultimate consumer to be correctly informed as to the extra financial burdens he must carry, should be considered.

If a fair international compromise is to be reached, it would seem that the country which prefers the "best plan" should not be penalized by being forced to carry the extra costs imposed by a less economical plan. Therefore, if for any reason of sentiment or otherwise either country should prefer a more expensive plan, the extra capital cost and the extra operating expense should be determined and stated. In this way, both countries will understand in advance what extra cost is incurred because of the adoption of the more expensive plan. Such extra cost could be met by proper allocation of cost between the two countries or proper division of the power output.

The International Boundary should be adjusted, if necessary, so that the Canadian power-house will be on the Canadian shore, regardless of the adopted plan. Such adjustment of boundary could also well equalize the loss of agricultural land by the two countries. Only the southern part of Canada is climatically adapted to intensive diversified agriculture and it is quite natural to resent the greater sacrifice of land on its part.

Under such procedure, the public mind will be assured of the superiority and fairness of the adopted plan, the opposition to this development will largely disappear, and the citizens of both countries will give to this great project the whole-hearted support it merits.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ESTIMATING THE ECONOMIC VALUE OF PROPOSED HIGHWAY EXPENDITURES

Discussion

BY MESSRS. W. W. CROSBY, ROGER L. MORRISON, AND SAMUEL B. FOLK

W. W. CROSBY,[3] M. AM. SOC. C. E. (by letter)[3a].—A tenet, often advanced from certain quarters, is that the return in the form of economic transportation is the sole measure of justification for road improvement. Of particular interest is Professor Agg's apparent admission that this sordid tenet is not always to be accepted, but there may be some uncertainty as to just how far he means to go in emphasizing it. He states in the "Synopsis" that, excepting certain works to promote safety, most construction work of State Highway Departments and of cities consists of projects that should go forward only when they are justifiable on economic grounds. He states further that most of the major improvements do not result in a reduction of the annual cost of maintenance and must be considered on broader grounds, in order to justify the necessary expenditures. On the other hand he declares that in most cases the ultimate purpose of highway improvement is to permit transportation at the minimum cost.

Much seems to depend on just what Professor Agg intends the word "economic" to mean. As the writer attempted to suggest in a previous paper,[4] it may be used with varying meanings. Professor Agg seems to mean it to be interpreted narrowly as "saving." At the meeting of the Highway Division of the Society in Chicago, Ill., on June 29, 1933, E. J. Mehren, M. Am. Soc. C. E., pictured[5] the ideal highway system as one that is adequate for the traffic in each area served, properly marked, fitted with adequate signaling devices, with grade crossings eliminated, with footpaths, night lighting, landscaping, and with comfort stations. According to Mr. Mehren,

NOTE.—This discussion of the paper by Thomas R. Agg, Assoc., M. Am. Soc. C. E., presented at the meeting of the Highway Division, Atlantic City, N. J., October 10, 1932, and published in September, 1933, *Proceedings*, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Cons. Engr., Coronado, Calif.

[3a] Received by the Secretary October 9, 1933.

[4] "Economic and Engineering Problems of Highway Location," by W. W. Crosby, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 1007.

[5] *Civil Engineering*, August, 1933, p. 458; and *Roads and Streets*, August, 1933.

engineers and other officials are projecting their minds into the future and preparing the way for safer, more comfortable, and less hampered traffic.

How can such a vision materialize if engineers are bound by acceptance of the illiberal dogma based on cold economics? Nevertheless, that materialization is yearly, if not daily, nearer reality in many places.

Charles B. Breed, M. Am. Soc. C. E., in discussing[6] the cost of highway transportation, has pleaded for a clear distinction between the cost and the value of transportation. Both factors must be appraised properly to determine whether the expense is justified. A highway, according to Professor Breed, may be entirely commercial, or it may serve partly for school transportation and for protection as well. Quoting:

"It may be impossible to express these values in money, yet these may be so obvious as to carry unanimous public approval of a policy involving a road cost well above any saving that can be computed in dollars and cents."

Thus, according to the authority quoted, the justification of most large highway improvements will not be found entirely in their effect on transportation cost, but in their influence on transportation values, of which cost is only one element.

Discussing a paper by the late Stephen R. Mather on "Engineering Applied to National Parks," H. K. Bishop, M. Am. Soc. C. E., likewise deplored[7] a tendency to think too much of economy in road construction and too little of the preservation of beauty.

The writer adds his endorsement to the foregoing idealistic concepts and submits that the results obtained by the Westchester County (New York State) Park Commission already emphasize the importance of certain "intangibles" connected with any highway improvement, which more and more demand consideration in questions of highway engineering.

The paper is based on logical reasoning within certain, perhaps narrow, monetary limits. One might dispute some of the author's assumed figures, and question the local acceptability of some of his "averages"; but probably this would lead to no results of real value, because of the neglect of the "intangibles" in the end. If he could evolve equations with the "intangibles" included, and a means for substituting "values" in them, highway planners might then welcome the general acceptance of a mathematical solution of some of the most troublesome of highway questions.

ROGER L. MORRISON,[8] M. AM. SOC. C. E. (by letter).[8a]—This paper is especially timely just now (1933) when those favoring "road-building holidays" are considering only the amount of their taxes, while most of those favoring more road building offer no reason for it except increased employment. Both sides practically ignore the important economic questions involved.

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[6] *Proceedings*, Highway Research Board, Vol. 12, Pt. 1, p. 32.

[7] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1193.

[8] Prof. of Highway Eng. and Highway Transport, Univ. of Michigan, Ann Arbor, Mich.

[8a] Received by the Secretary October 18, 1933.

One reason for this, perhaps, is the necessarily complicated formula required in order to include all factors for an indefinite time into the future. Actually, of course, the most painstaking mathematical calculations are no more accurate than the basic data assumed, such as the life of the different parts of the highway and the everchanging costs of motor-vehicle operation.

It would seem, therefore, that reasonably accurate results might be obtained by considering only the period covered by the life of the first pavement. If this is assumed to be, say, 20 years, then the value of the highway at the end of that time can be estimated, and the difference between that value and the first cost is the depreciation, while one-twentieth of it is the annual depreciation. Periodic renewals of such parts of the highway as may not last 20 years may be allowed for in the sum estimated for annual maintenance. Under this method only the three items of depreciation, maintenance, and interest, need be considered in computing the annual cost.

It is freely admitted that this method is subject to attack from the standpoint of inaccuracy, and, also, that if an attempt is made to include in a formula all the operations involved, little may have been gained in the way of simplicity. For practical purposes, however, it does not seem necessary to include all these operations in a formula, and the method may possibly be more easily understandable, and hence more likely to be generally used, than the formal method given by Dean Agg. His accurate formula, however, is the standard by which any attempts at simplification should be judged.

More serious than the differences between the various methods that may be used in making calculations is the scarcity of up-to-date information as to operating costs over various surfaces, particularly the so-called low-cost types which have been developed recently. With the enormous sums involved it would seem that the proper solution of these economic problems is sufficiently important to justify every highway department in collecting data on the cost of vehicle operation over different types of surfaces.

A word of caution appears to be desirable in connection with attempts to evaluate a highway system by means of an economic analysis. If an earth road, for instance, is replaced by a pavement, there may be an average saving of nearly 2 cents per mile for all vehicles that would use the earth road if the pavement were not there, but it does not follow that, where extensive traffic has developed after a system of pavements has been built (and largely because the pavements were available), there is a similar saving for all vehicles using the improved system. Without the pavements much of this travel would be by rail, or would not be undertaken at all, so that, to this traffic, "savings" indicated as computed herein, are largely imaginary. It is "proposed" highway expenditures which Dean Agg discusses, not expenditures several years in the past.

SAMUEL B. FOLK,[9] ASSOC. M. AM. SOC. C. E. (by letter).[9a]—This paper has done much to clarify current ideas, and to summarize a tangible method for attacking the problem of justifiable costs of improving a highway. It is

[9] Asst. Prof. of Mechanics, Ohio State Univ., Columbus, Ohio.
[9a] Received by the Secretary October 26, 1933.

somewhat unfortunate, however, that more emphasis was not placed on some parts of the analysis.

The basic principle of,

$$\text{Highway Transportation Costs} = \text{Highway Costs} + \text{Vehicle Costs}$$

should be somewhat elaborated and qualified by provisions; that these costs shall fall equitably on those able and willing to pay; that collections must be made promptly; and, that payments must be sufficient to retire outstanding obligations when due. At times, there is a tendency to refer to the cost per ton-mile or per vehicle-mile that would justify improvement in the aggregate, omitting from consideration the painful fact that a few constant and important users of the new road will profit by decreased vehicle-operating costs, while many occasional users save little in operating costs and are required to meet the heavier interest and amortization charges on bonds retired from taxes on abutting or on adjacent property. This is an unfortunate but archaic system followed under some State laws; and while this phase of highway economics is beyond the scope of Dean Agg's paper, it is hoped that engineers will rise to sponsor an equitable method of financing improvements as well as a justifiable program.

The assumption that from 2.5 to 3.3 cents per mile would be reasonable for an allowance due to shortening of distance seems fair and is borne out by many studies.[10] The same may be said for the 0.8 cent per automobile mile saved in changing from one type of surface to the next higher. However, current notions of economics have undergone such serious upsets in recent years that it might be well to apply a "factor of safety" to any calculation involving the saving per car-mile multiplied by the total traffic per year. Such studies usually assume a constant annual traffic that is probably nearly correct for commercial vehicles; but a lower type of roadway discourages pleasure driving and thereby affords a real saving, whereas a higher type of surface encourages more pleasure driving (which operates at a lower cost) than a poor road. It has not been shown from studies, however, what percentage of traffic is chargeable to business and what percentage to pleasure, which would otherwise not be undertaken and which, therefore, cannot be credited to savings. The same philosophy might be applied to the saving derived by shortened distance as with savings accruing from operation on higher types of roads. Dean Agg has done well in pointing out the ease of arriving at such a conclusion by fallacious adaptation of the results of careful investigations.

[10] *Bulletin 91*, Iowa Eng. Experiment Station, Ames, Iowa.

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DISCUSSIONS

STABILITY OF STRAIGHT CONCRETE GRAVITY DAMS

Discussion

BY H. DE B. PARSONS, M. AM. SOC. C. E.

H. DE B. PARSONS,[11] M. AM. SOC. C. E. (by letter).[11a]—The writer agrees with Mr. Henny that shear is a more important factor than sliding, and that consideration of a safety factor should be based more on the former rather than on the latter. With good, hard rock, properly prepared and roughened and with all loose parts removed, it is difficult to conceive that true sliding could occur. Under such conditions, sliding at the foundation plane could take place only by shearing some of the rock, and the shearing strength of good rock exceeds that of concrete. With a rock that is likely to soften under continuous water pressure, the danger is generally greater at the side abutment joints than under the central base of the dam. Security against such weakness is best provided by a suitable design.

If possible, the surface of a rock foundation should be graded upward in the down-stream direction. Such grading gives a dam additional resistance to movement down stream, and, also, permits a component of any uplift pressure to act up stream against the hydraulic pressure.[12]

On a pervious soil foundation, there is danger that a masonry dam may slide, although, with a properly designed base, any sliding of the structure would carry some of the foundation material with it; that is, any sliding would cause shearing of the soil. For such a foundation, the writer advocates a wide base, and such a sectional profile of the dam structure that the reservoir water will add weight to hold the dam in position. This can be done by designing the up-stream face on a rather flat slope.

The intensity per unit of area of the uplift pressure on the base of a dam may be greater with some classes of good rock foundation than with broken or fissured rock or pervious soil. Under the latter conditions the hydrostatic head may be lessened since the water has a freer exit from under the

NOTE.—The paper by D. C. Henny, M. Am. Soc. C. E., was published in September, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[11] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

[11a] Received by the Secretary October 30, 1933.

[12] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 185.

base. Nearly all recorded uplift intensities are less than that represented by an even gradient from reservoir surface to tail-water;[13] therefore, Mr. Henny's proposition to estimate the average intensity at 0.50 H should be safe. In designing dams, especially those that are high and important, it is well to make assumptions that tend toward safety.

The effective area of the base on which this intensity of pressure acts cannot be predicted with absolute certainty in the case of rock foundations. As the rocks present many variable conditions, it is only by repeated investigations and the increased knowledge thus gained, that effective area may be approximated in percentage of total area, so that fair accuracy can be obtained.

The uplift pressure will not be uniformly distributed. If the effective area on which the uplift acts is, say, 35% of the total, the effective part of any selected part of the total may be more or less than this percentage.

Grouting under the up-stream part of the base of a dam will not remove entirely the hydrostatic uplift from under the structure, no matter how well it may be accomplished. Uplift has been found under dams that have had the foundation rock well grouted.

When the foundation is a pervious soil, uplift can be expected to act on the full area of the base with an intensity diminishing from the up-stream corner to the down-stream corner. While the uplift at the down-stream corner will correspond to the tail-water head, the uplift at the up-stream corner will be considerably less than that due to reservoir head, depending on the efficiency of the cut-off. There should not be a cut-off or obstruction of any kind at the down-stream part of the base that would prevent the water from having a free exit. If there is such obstruction the hydrostatic uplift will be increased.

Mr. Henny has brought out very clearly the necessity for making an allowance for uplift force in the design of a dam, on either rock or pervious soil. For important dams, this allowance should be generous rather than meager. Considerable information of actual uplift intensities under dams is now available. The writer believes that foundation materials could be classified and that to each could be assigned a safe assumption of uplift intensity. While the lower limit would be the tail-water head at the down-stream corner of the base if the tail-water surface was above the base, and zero if the tail-water was below it, the maximum limit, taken at the up-stream corner of the base, would vary from a fraction of the reservoir head above the base to full reservoir head. Thus, for a specific class of foundation material, the maximum intensity could be taken at some figure between $f\,H$ and H, diminishing uniformly to tail-water head if above the base, or to the elevation of the down-stream corner if below. In like manner, the percentages of effective areas of the base could be classified with reference to the character of the foundation material. Such assumptions, for present purposes of design, could be corrected to closer classifications from time to

[13] "Hydrostatic Uplift Under Masonry Dams," by H. de Parsons, M. Am. Soc. C. E., *Proceedings*, World Eng. Congress, Tokyo, 1929, Vol. IX, p. 295.

time by additional observations and researches, and increased knowledge obtained therefrom.

The writer avoids the expressions "heel" and "toe." Dams are now designed with such a variety of shapes, that these words have become meaningless as they have lost their pictorial value. Uniform terminology should be encouraged, and better expressions are "up-stream corner" or "down-stream corner," which leave no uncertainty as to which corner or edge of the base is meant. Similarly, it is customary to refer to "length" as the axial measurement of a dam from abutment to abutment, although Mr. Henny has used the word as meaning the "width" of the base. Engineers should set an example in the use of correct expressions when writing on technical subjects.

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DISCUSSIONS

THE SURVEYOR AND HIS LEGAL EQUIPMENT

Discussion

BY MESSRS. RAY H. SKELTON, WILLIAM BOWIE, R. ROBINSON ROWE, AND WALTER H. DUNLAP

RAY H. SKELTON,[78] M. AM. SOC. C. E., (by letter)[78a].—The law of boundaries is extremely complicated and characterized by endless exceptions which Professor Holt has not stressed. However they are mentioned, and it is important to note them, because all principles laid down by the Court regarding the control of conflicting elements are rules of construction, and as such may be departed from when their rigid application defeats the true purpose of the survey, namely, the establishment of the line in its intended position.

Although the paper presents little material that is new to a competent surveyor, its publication is justified because it brings to the attention of the young engineer the "art of surveying." The field of engineering has developed so extensively in the past generation that the school curricula are crowded and, consequently, the time devoted to the fundamental subjects has been curtailed. Unless a college offers an opportunity for a student to major in surveying, it is impossible for an instructor to teach more than routine procedure. While it is not desirable that the time be taken to develop skill, it is important that the legal principles of boundaries be brought to the attention of the student, and any course that does not contain a discussion of the "art of surveying" is incomplete. Professor Holt's paper is an excel lent assignment for such an exercise, and its inclusion in a course should give a student a new conception of the practice of surveying, and a foundation upon which to build if he follows the fascinating field of boundary work.

The writer regrets that Professor Holt did not include in his discussion of "A Cardinal Principle", the decision of Judge A. R. Savage, [79] of Maine,

"The cardinal rule for the interpretation of deeds and other written instruments is the expressed intention of the parties, gathered from all parts

NOTE.—The paper by A. H. Holt, M. Am. Soc. C. E., was published in September, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[78] Associate Prof., Coll. of Eng., Univ. of Maryland, College Park, Md. Prof. Skelton died on October 29, 1933.

[78a] Received by the Secretary, September 27, 1933.

[79] Perry *v.* Buswell, 113 Maine, 399, 94 Atl. 483, 484.

of the instrument, giving each word its due force, and read in the light of existing conditions and circumstances. It is the intention effectively stressed, not merely surmised. This rule controls all others."

and that of Judge S. W. Sanderson,[80] of California,

"* * * the only rule of much value—one which is frequently shadowed forth, but seldom, if ever, expressly stated in books—is to place ourselves as nearly as possible in the seats which were occupied by the parties at the time the instrument was executed; then, taking it by the four corners, read it."

These citations are admittedly vague, but they do define the foundation upon which to base an investigation for the determination of the true location. The sundry rules of construction are merely guide-posts which may or may not lead one to an understanding of the "expressed intention of parties," and are to be disregarded if they point elsewhere.

The writer once based a survey on a width of street as 60 ft, whereas the legal width at the time of the conveyance was 50 ft, because he found documentary evidence on record that the parties to the transaction had erroneously regarded the street as 60 ft wide. Consideration of this error enabled him to visualize "the seats which were occupied by the parties at the time the instrument was executed," and to lay a boundary in conformity with the deed then written and in harmony with the landmarks of to-day. Frequently, the parties sit in the wrong seats because of errors of omission. It is a too common practice to transfer properties without a survey, and frequently former descriptions are copied without regard to changes that affect them. A street is widened and in a conveyance subsequent to the widening a description that is correct with regard to the former side line of the street is erroneously copied. If one follows the rule that measurements from street lines are to be interpreted as of date of deed,[81] he will not arrive at the correct line, and it is only by giving due regard to the party's error of omission that he can establish a boundary in conformity with their intention.

Unfortunately, landmarks are transitory, and as they disappear the "seats" decay, and difficulties in interpreting the expressed intention develop. A knowledge of local history,[82] therefore, is an important part of a surveyor's equipment, and no surveyor, however competent otherwise, should attempt work in a strange community. Furthermore, in writing a deed description, it is advisable to recognize the temporal nature of all things and to base the description on the most permanent control available, so that future generations can establish beyond a shadow of a doubt the expressed intention of the parties. It is regrettable that in many cities lawyers will start a description from a theoretical point which has no existence, despite the availability of a good control.

While it is true that little use has been made of the precise surveys of the United States Coast and Geodetic Survey it is encouraging to observe

[80] Walsh *v.* Hill, 38 Cal., 481, 487.

[81] Tebbets *v.* Estes, 52 Maine, 566; Syracuse *v.* Cook, 187 App. Div. 578, 176 N. Y. S., 396.

[82] "City Surveying for Title Examination," by Edwin P. Clark, Brooklyn Title Guarantee & Trust Co., 1921. (A local history of Brooklyn, N. Y., of interest to surveyors.)

that a beginning has been made. The activities of the various regional planning commissions are extending the use of exact control into the suburbs of the larger cities. With the co-operation of Mr. Edwin P. Clark, of the Brooklyn Title Guarantee and Trust Company, the engineers in charge of sub-divisions at Montauk Point, Long Island, adopted a geodetic station as the basis of their local control. A case has come before the Lower Courts involving the marketability of a title in one of these sub-divisions. The plaintiff claimed the description was such that he could not locate his property, but the Court held that inasmuch as any competent engineer could establish the lines, the title was good and marketable. The case suggests the desirability of following the technical description referred to the precise control, by a separate one which will identify the property for the layman.

Recently, the writer has been considering the location of property lines disturbed by an earthquake. This question is by no means academic; Charles Derleth, Jr., M. Am. Soc. C. E., in describing some effects of the San Francisco earthquake reports[83] that a property fence crossing a pipe line about 10 ft to the south of a certain rupture was offset 7 ft along the fault line and that the two pieces of the fence remained straight and parallel to their original direction.

Inasmuch as an earthquake disturbs the boundaries themselves, its possibility cannot be regarded as a weakness inherent to geodetic control alone and used as an argument against the adoption of such control. The question the writer desires to raise is: "How are boundaries adjusted after such major disturbances?" In the open country the matter is of little consequence, but where the fault crosses a city, property of high value is involved. A search of law reports has not as yet revealed a single case that has come before the Courts. Undoubtedly, the property owners recognized the catastrophe as "an act of God" and were ready to accept reasonable compromises. In the time of such disaster, relief commands the first efforts of all, and re-establishment of such essential services as water and sewer demand the attention of the engineer, but before reconstruction can begin, disturbed street and property lines must be adjusted. There is no time for extensive situation surveys and careful consideration or the immediate recording of all adjustments. Information as to how this secondary but, nevertheless, important problem was treated, would be a valuable addition to the paper.

WILLIAM BOWIE,[84] M. AM. SOC. C. E. (by letter)[84a].—Every engineer, lawyer, and property owner should feel under obligations to Professor Holt for the help that he has given them in this paper. Much of the confusion regarding the boundaries of public and private property could be avoided by a careful co-ordination of the legal and engineering aspects of boundary location.

A phase of the boundary problem of great importance was mentioned in the concluding paragraphs of the paper where it was suggested that boundary monuments be tied into the precise surveys of the U. S. Coast and Geodetic

[83] *Engineering News*, Vol. LV, No. 20 (May 17, 1906), pp. 548, 550.

[84] Chf., Div. of Geology, U. S. Coast and Geodetic Survey, Washington, D. C.

[84a] Received by the Secretary October 10, 1933.

Survey. Professor Holt's suggestion can be followed now (1933) to far greater extent than would have been possible a few years ago. More triangulation for the National fundamental net has been done during the past ten years than in the whole previous life of the nation. This increase has been due entirely to the greater need for accurate control surveys. When there was a lack of demand on the part of the people, Congress and the executive officials of the Federal Government did not feel justified in making increased appropriations for the work. Now, however, they recognize that control surveys are a public work of great value, that should be rushed to an early completion.

There now are, in the United States, approximately 40 000 miles of arcs or chains of triangles. The widths of these arcs vary. In flat, heavily wooded country the average width is from 6 to 8 miles. In mountainous areas the width of modern triangulation is approximately 20 miles. The average width for the entire country may be safely assumed to be 9 miles That means that 360 000 sq miles lie within the limits of the triangulation itself. There are very few places inside the arcs that are more than 10 or 12 miles from a triangulation station. The strips of land lying on each side of an arc, out to a distance of about 10 miles, are within easy reach of the triangulation stations. Therefore, there are approximately 1 000 000 sq miles, or about one-third of this country within short distances of precise triangulation stations.

The boundary situation could be improved if each State or each county would run lines of traverse between contiguous triangulation stations with enough cross-lines to form a traverse net. Traverse lines could also be extended over the 10-mile strips on either side of the arcs of triangulation. Monuments should be set at frequent intervals along these lines. The traversing of the area now controlled by triangulation could be undertaken at once and could be expanded to other areas as new triangulation is done.

There is no difficulty in running a traverse with an accuracy of at least as good as 1 part in 10 000. This accuracy is, in general, sufficient for any farm property boundary survey and for property surveys within the limits of a small settlement, such as a village or small town. The engineer with a steel or invar tape and a surveyor's transit, reading even to single minutes, can readily obtain an accuracy of 1 part in 10 000 in his traverse. This has recently been demonstrated by traverse parties in North Carolina, working under the direction of the State Highway and Public Works Commission.

The Coast and Geodetic Survey has recently taken a long step forward in adopting single-plane co-ordinate systems for the States of North Carolina and New Jersey and for the area of Long Island. In New Jersey, the scale factors have a maximum value of 1 part in 30 000, except for a very small part of the area in the southwestern part of the State. On Long Island, the maximum scale factor is somewhat smaller than 1 part in 100 000. The greater expanse of North Carolina results in a somewhat larger factor for that State, but one that is quite satisfactory to the State Highway and Public Works Commission which co-operated in the adoption of the system.

There are many engineers who hesitate to use triangulation and traverse data which are based on spherical co-ordinates, that is, on latitudes, longitudes, and azimuths (or true bearings), but who find no difficulty whatever in using the results of such triangulation when the positions are computed on a rectangular system. Rectangular systems will undoubtedly be adopted for other States but, in many States, it will be necessary to have more than one system because of the great width involved. A State with a width of less than 158 miles in the east-west or north-south direction can have a single co-ordinate system with scale factors not greater than 1 part in 10 000.

The tying in of property surveys, whether public or private, to the triangulation system of the country safeguards the rights of the property owners. There will be little or no difficulty in recovering, at any time in the future, a property line connected to control surveys even if all the monuments that were originally set, have been destroyed. Markers may be destroyed, but co-ordinates are permanent as long as a few stations of the triangulation net of the country remain in place. The mortality of triangulation station marks is exceedingly small, and the property owner may rest assured that, when his lines are tied into the National net, he and his heirs will always be able to recover their lines.

In the State of New York the referencing of property surveys to the National triangulation system has recently been given legal approval through a decision of the State Supreme Court.

R. Robinson Rowe,[85] M. Am. Soc. C. E. (by letter)[85a].—The practice of surveying has enjoyed unusual prominence in recent publications of the Society, and there is need for much more. The comparison[86] by Mr. M. L. Greeley of the high standards and standing of the profession in Canada, with the Kentucky decision of 1920 that surveying was a trade, is not a great exaggeration, unfortunately, of the actual status. Professor Holt's timely paper suggests not only a contributing cause, but also some of the remedies. Since the Profession of Civil Engineering includes surveyors, inevitably time will level them—either to the rank of professions or to the file of the crafts. Hence, the interest of all civil engineers, even though many do not practise surveying.

The great demand for resurveys of residential lots during periods of active construction has presented the opportunity for unqualified men to establish themselves as surveyors. Experience as instrumentmen under competent party chiefs may have given them sufficient aptitude in the use of the instrumental equipment, but it did not often give them proficiency in the use of "legal equipment." Their surveys were ordinarily dependable, if acceptable monuments were evident and the original survey well done. Usually, they depended upon offset lines or witness corners of city or county surveyors and if, occasionally, they were too trusting in the infallibility of such officers, at least they enjoyed a good alibi. When such evidence failed, still the survey was made; no problem was too difficult for a solution. North was north, and

[85] Asst. Engr., San Francisco-Oakland Bay Bridge, San Francisco, Calif.

[85a] Received by the Secretary October 19, 1933.

[86] *Civil Engineering*, September 1933, p. 510.

100 ft was 100 ft, in the description and on the ground. These surveyors knew no law except that of single proportion and no logic except "confidence begets confidence."

Their overhead expense was nominal, based on education gained on the job, equipment rented or borrowed, residence telephone with a business classification, and assistants paid by the hour. When the depression came, they reduced prices and solicited business to keep busy. Since careful surveyors would not compete with such tactics, the business of the latter was soon limited to a small discriminating clientele, surveys of litigated property, etc.

Conflicts between such careless surveys, price-cutting tactics, solicitation, and the type of man who now claims to be a surveyor, have made a trade of lot resurveying and kindred work, yet most of these men tried to do the right thing. Ignorance of the law, rather than inaccurate field work, caused the conflicts; ignorance of ethics, rather than lack of conscience, led to price-cutting and solicitation.

The remedies consist of eliminating the unfit and educating the fit in the legal equipment and ethics of surveying; compiling for all surveyors, the essence of the law governing surveys, not only for Minnesota and Iowa, but for all States; and obtaining such benefits as laws may make available to assure an increasing respect for the profession. The following suggestions are offered as initial steps toward definite remedies:

First.—A code or standard of practice for surveys and resurveys should be compiled and published, including ethics, common law, and minimum requirements for accuracy and monumentation. The code might determine the availability of one surveyor's notes to another, or require the making of a public record of the restoration or perpetuation of monuments, or even require perpetuation or witnessing of monuments in danger of obliteration.

Second.—Digests of common law, statutes, and decisions should be compiled, showing for each State, if different, the statute of limitations, leading or ruling decisions on adverse possession, acquiescence, riparian boundaries, proportion, and all other matters which have led to litigation with admitted facts, and, also, liability of surveyors for damages resulting from mistakes or trespass.

Third.—An investigation should be made of the possibility of legalizing arbitration as a means of determining boundaries, where the decision must be based on facts rather than on law. Such procedure would save money, time, and tempers.

Fourth.—The desirability of assuring the accuracy of surveys should be ascertained by an investigation of legal phases, probable costs, possible demand, and ethics involved. Buyers of real property usually insist upon title guaranty or insurance, but presume that possession is proof of agreement between deed description and lines on the ground as indicated by the seller. Further insurance might be demanded if available.

The cost would be that of a careful survey, premium on the risk, and a nominal charge for plat and certificate. The writer can conceive some advantages to this, namely, that insurance cost would be determined by value

of property, its proposed use, and particularly by the development up to the property line, as well as the ability of the engineer (based at first upon reputation and later on merit rating). Hence, the careful scrutiny of the insurance companies would eliminate irregular surveyors from this class of work by setting high premiums. On involved surveys, the insurer might require consulting surveyors or attorneys to review the work. There would be a tendency of owners ordering uninsured surveys to prefer the services of surveyors qualified to make insured surveys.

The important thing is to raise the standards and standing of the surveyor. Professor Holt is well within his rights in stressing many apparently simple principles that should govern the conduct of resurveys. Until these and many other principles of law and ethics are recognized by surveyors, the profession will remain a trade.

WALTER H. DUNLAP,[87] ASSOC. M. AM. SOC. C. E. (by letter).[87a]—It is difficult to add to, or subtract from, Professor Holt's excellent exposition of the legal principles pertaining to boundaries. The discussion herewith aims to present an historical background to account for, and to illustrate, the principles set forth by the author.

Doctrine of Adverse Possession.—In the early days of Virginian history, Charles II of England freely made extensive grants of wild land in America to favorites and as rewards for public services, regardless of the fact that such land was already occupied by others to whom he had previously granted it. In 1684, the rapacious and unprincipled Lord Culpepper, whose chief interest in Virginia was exploitation for his own profit rather than benefit to the growing colony, was confirmed by the Crown in his title to the Northern Neck of Virginia, being that slender peninsula lying between the Potomac and Rappahannock Rivers.

At that time little was known of the country west of the fall line (Fredericksburg), but in 1716 the public spirited Spottswood led his Knights of the Golden Horseshoe across the Blue Ridge Mountains and discovered the Shenandoah Valley, whereupon Lord Fairfax, grandson of Culpepper and heir to the Northern Neck, claimed all the land between the two rivers all the way back to the source of each, which was quite an indefinite boundary. Fairfax later established himself in the Valley at "Greenway Court," near Winchester, and proceeded to dispose of his lands in parcels called "quit claims." His surveyor was George Washington.

In the meantime, others had settled on the land. About the time (1607) of the first permanent colony in America at Jamestown, James I also planted in the northern part of Ireland a colony of picked men and women from Scotland, and by the beginning of the Eighteenth Century the colony had become so strong and its textile industries so prosperous as to engender violent jealousy on the part of English manufacturers. Religious intolerance and economic persecution drove many of these Presbyterian Ulstermen to emi-

[87] Asst. Engr., Public Utilities Comm., Dist. of Columbia, Washington, D. C.

[87a] Received by the Secretary October 30, 1933.

grate to America where they pierced through the seaboard settlements and became a kind of buffer State against the Indians. They stopped in Pennsylvania long enough to learn the wisdom of purchasing rather than grabbing the land from the Indians; but this policy was not so easy to follow because the Valley of Virginia was the common hunting ground, and, consequently, the battleground of several warlike tribes.

Such were the Scotch-Irish settlers whom Lord Fairfax found on "his" land. Theodore Roosevelt calls them the first real Americans, because long before the Cavalier settlements in Tidewater Virginia had ceased to give full loyalty to the Crown, these rugged settlers, exposed to Indian warfare from which they were entirely unprotected by the Crown, recognized no earthly sovereign. When Fairfax undertook to claim "their" lands, stubborn disputes followed, which were carried on in the Courts for a full hundred years.

The present doctrine of Adverse Possession and Squatter's Rights is undoubtedly based in considerable measure on the law as developed in those long drawn-out legal battles between the Cavalier proprietors and the Puritan squatters. Thomas Jefferson has sometimes been called the father of American independence and the Shenandoah Valley its cradle.

Retracing Old Boundaries.—Land titles in Eastern Kentucky present some difficult and interesting problems in the application of the principles recited by Professor Holt.

During the last decade of the Nineteenth Century and the first decade of the present century, coal land organizers in Eastern Kentucky were very active in the business of acquiring from the small farm owners, titles in fee simple or for mineral rights to as many adjacent parcels as possible in the effort to block up areas sufficiently large to justify large-scale mining operations. One of the most successful of these organizers was the late John C. C. Mayo, of Paintsville, Ky.

In 1906, Mr. Mayo outlined an area of about 30 000 acres mostly in Johnson County, put lawyers to work abstracting titles, and organized a corps of surveyors and draftsmen, of which latter group the writer was a member. Transit traverses were carefully run up the main streams and through the gaps to form great loops, which, in turn, were divided into smaller loops by tie lines. Latitudes and departures were computed for closure and for co-ordinates, which latter were used in mapping. The metes and bounds of every separately owned tract were then run out by land surveyors with compass and tape, and tied into the transit traverse for control.

The survey was platted on sheets of mounted paper 5 ft wide by about 12 to 15 ft long, on a scale of 400 ft to an inch, and areas were taken with a planimeter. At any time an owner was inclined to sell, and a price per acre was agreed upon, the deed containing the surveyor's description could be written and the deal closed within an hour.

In many cases it was found necessary to have the surveyors run out not only the lines called for in the deed, making notes of fences, clearings, years of occupancy as claimed, etc., but also to run out the underlying patents as well. These patents merit a short description.

About 1775 white men began to settle south of the Ohio River, and for the next twenty-five years, or more, the authorities of Colonial Virginia and the new State of Virginia granted to various persons letters patent to large, indefinitely described, unsurveyed or partly surveyed tracts of land in the eastern mountainous part of what became in 1792 the State of Kentucky. The new State likewise granted patents for such tracts, neither knowing nor greatly caring that much of the land was already covered several layers deep by such grants, or that *bona fide* settlers actually occupied it to an increasing extent.

About 1800, these mountain settlers began to take out patents covering the narrow meandering strips of bottom-land they had cleared. These grants, known as "blacksnake surveys" from their shape, were followed some years later by larger surveys made for the purpose of including the mountain land to or beyond the top of the ridge.

The practise of the old surveyors in making these surveys is well authenticated. They actually measured the metes and bounds along one side of the clearing, using standing trees as corners, which they marked and described. At the beginning tree and at the last tree of this measured traverse they usually sighted the compass roughly in the direction desired, recorded the bearing, and went back to the house. In the "office" they platted this base line and drew other lines which they judged would enclose the areas desired. In writing the description they protracted and scaled the lines not measured in the field, and described each of the unsurveyed corners as being marked by a stake. The description always closed with the words "containing blank acres more or less."

When these old descriptions are re-run in later years, one or more of the marked trees of the base line can often be found, but the distances almost invariably fall appreciably short of those recorded by the early surveyor. The problem of how to adjust the old stake calls to the corrected base then arises.

A basic principle of land surveying is that the description must be reasonably construed. It is easy enough to accept the more accurate measurements of the recent survey as correctly describing the line from one known marked tree to another, and to swing the entire survey in azimuth enough to offset the change in magnetic declination, but it is more difficult to set up a solution that will adjust the stake calls more reasonably than any other that may be proposed by an opposing claimant.

The usual procedure is to plat the original description and—if it closes reasonably well and if the acreage approximates that intended to be enclosed by the original proprietor—to claim that the stake calls were not fictitious, but real lines, surveyed on the drawing-board rather than on the ground, and fitted more or less truly to the area in the minds of the surveyor and his employer. This reasoning and the principle that lines may be run backward as well as forward permit the entire error to be thrown into the first stake call; or, if other circumstances indicate that it is more reasonable to throw it into the line from the last stake to the beginning corner, that can be done.

It is a difficult problem at best; it is not an academic one at all, because such surveys are very, very numerous, and the coal underlying the surface is valuable. In tracing such descriptions the surveyor must proceed with confidence, because one or more of the adjoining land owners usually accompany the survey party, and the men of the Southern Appalachians are notoriously contentious for their rights. Land disputes constitute the major cause at the root of most of the family feuds among them.

The old surveyor usually measured directly along the slope rather than on the horizontal, and was not too careful in chaining to keep to the straight line connecting the points. As a consequence there was much overlapping of surveys from opposite sides of a ridge. Some surveyors kept the chain tally by tying knots in the leather thong at the end of the chain, and if one or more knots were lost in dragging the chain through the bushes, error resulted.

Apologies must be made to the old surveyors whose mistakes are emphasized herein for the purpose of describing the problems presented to the land surveyor of the present. Most of their work was as good as could be expected with the instruments and help at hand, done as it was at a time when a good jack-knife sufficed to purchase a number of acres of land that covered coal deposits which are now worth thousands of dollars.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DEFORMATION OF STEEL REINFORCEMENT DURING AND AFTER CONSTRUCTION

Discussion

BY MESSRS. WILLIAM RUSSELL DAVIS, AND J. R. WORCESTER

WILLIAM RUSSELL DAVIS,[11] M. AM. SOC. C. E. (by letter).[11a]—By emphasizing that the shrinkage of concrete over a long time should be kept constantly in mind by designers of structures in which concrete is used, this paper is made a most valuable contribution.

The writer has observed the results of long-time shrinkage in the increasing deflection of concrete beams that have reinforcement, for the most part, in the bottom only. The effect of such shrinkage has been the distortion of door frames, followed by the binding of doors, and the cracking of partitions. It is evident that the shortening of the unreinforced top, by shrinkage, must be greater than the shortening of the reinforced bottom, and that this relative change in length must be accompanied by an increased deflection which does not impair strength, but is a cause of anxiety to the owner of a building when doors bind and partitions crack. These results can be avoided by using adequate steel beams under partitions, or by using, in the top of reinforced beams that carry partitions, an area of reinforcement approximately equal to that used in the bottom. In fact, since the greater the percentage of steel the less the change in length from shrinkage, it may be predicted that, by using more steel in the top than in the bottom, the deflection will decrease as shrinkage takes place.

The author suggests that further experiments may yield data about shrinkage, which will lead to material changes in the formulas now used for the design of both beams and columns. The data cited in this paper appear to justify such an expectation in the case of columns, but the case for beams is very different. Commenting on Fig. 8, Professor Sergev states that some readings indicated compression in the bottom reinforcement of beams at 850 days and under "Calculated Theoretical Stresses," he states that "all beams under observation had no theoretical end restraint."

NOTE.—The paper by Sergius I. Sergev, Esq., was published in October, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[11] Cons. Engr., Albany, N. Y.

[11a] Received by the Secretary October 25, 1933.

The bending moment at the center of the beam must be counteracted by a resisting moment that can only exist if there is tension at the bottom. If there is no tension in the steel, the concrete must be in substantial tension despite the fact that it should not be considered as dependable in tension. In the case of the beams in question, it cannot be shown that the concrete was not stressed almost to the point of rupture at 850 days. These beams were observed under dead load only. The application of the live load might have caused rupture and produced a stress in the steel closely approximating the value given by the formula of the Joint Committee on Specifications for Concrete and Reinforced Concrete. That formula was never intended, and does not profess, to show the stress intensities that actually prevail prior to rupture of the concrete. The formula is predicated on the acceptance of the fact that concrete is not reliable in tension and that rupture is likely to occur. Just as long as concrete is unreliable in tension, such tests as those described in this paper, despite an excellent showing for the tensile quality of the concrete, cannot justify any modification of the beam formulas now in general use.

J. R. WORCESTER,[12] M. AM. SOC. C. E. (by letter).[12a]—A striking example is presented in this paper, of the effect of the shrinkage of concrete during the process of hardening and for some time thereafter. Although the fact that shrinkage, under normal building conditions, is to be expected, and although the likelihood of stresses in the reinforcement at variance with the usual design assumptions has been generally admitted, there has been little definite information, until recent years, as to the extent of such variance, the speed with which it may develop, or the time during which it may continue to increase.

As the author agrees, the results give insufficient data for the deduction of general rules for design, not only because of the comparatively small number of members tested, but particularly because the tests were not carried far enough to develop even the stresses under the design load, not to mention the stresses at the ultimate load. It is true, as suggested, that it seems inconsistent to base designs upon conditions of load that are not expected ever to be reached, but the impossibility of securing a desired factor of safety, without knowing what happens after a working load is passed, is apparent in the case of the beams described in the paper. It is evident that with the dead load alone the tensile strength of the concrete was not exceeded. No light is shed on what would have developed with a slight increase in load.

Professor Sergev's assumption that the beams were freely supported at the ends does not seem to be justified by the details shown in Fig. 1. It would appear from this diagram that the connection over the double columns was strong enough to develop the flexural strength of the column, and to have restrained the girder almost completely. The consideration of this restraint would have modified some of the calculations involving the beams. The maximum positive moment under the dead load would have been about 40 000 ft-lb instead of 71 500. Under this moment the tensile fiber stress in the concrete

[12] Cons. Engr., Boston, Mass.

[12a] Received by the Secretary October 28, 1933.

would have been only about 355 lb per sq in., a stress that might well have been resisted without fracture. It is obvious, however, that with a comparatively slight increase of load the concrete would have cracked, and the tension in the reinforcement would have been increased.

Perhaps the most surprising result obtained in this investigation was that "the rich mix [used in the lower section of columns] defeats the purpose for which it was intended." Apparently, this is due to the shrinkage being greater with the richer mix. The greater stress in the steel from this cause would probably continue as the load increased until the stress in the steel reached the yield point, but at that point there would be a re-adjustment, and the greater compressive strength of the richer concrete would have manifested itself.

There will be general agreement with the author's Conclusion 9, that "shrinkage should be considered as a property of concrete, and design equations should be altered to include the effect of shrinkage of the concrete both on the steel and on the concrete." In the matter of beams, where the effect of shrinkage is to increase the danger of cracks in the concrete, it is not apparent how the usual assumption that the concrete carries no tensile stress can be superseded.

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DISCUSSIONS

IMPROVED TYPE OF FLOW METER FOR HYDRAULIC TURBINES

Discussion

BY E. B. STROWGER, ESQ.

E. B. STROWGER[9], ESQ. (by letter).[9a]—The flow meter described by Mr. Winter seems to be in fairly wide use in the case of relatively recent hydraulic turbine installations. This has been brought to the writer's attention by the increasing number of turbine tests made by the Gibson method where a rating of casing piezometers is specified as a part of the work. Of the turbines tested under the direction of Mr. Gibson during the four years (1929–1933), nearly one-half have had casing piezometers installed. The inference that may be made from this trend is that some, or perhaps all, of the advantages pointed out by Mr. Winter for the installation of these piezometers, are becoming generally recognized.

In thirty-three recent tests by the Gibson method, the exponent, n, in the author's Equation (10), was found to vary from 0.484 to 0.544, the average value being 0.517. These data seem to confirm the author's Conclusion (5) which states that the value of the exponent, n, is substantiially 0.50 (see, also, Equation (9)). The relation between the computed value of P and the test value is expressed by the coefficient, C, of the author's Equation (3). In the tests cited this coefficient varied from 0.65 to 1.27, the average value being 0.99. These values roughly check Mr. Winter's experiments involving the range in C from 0.75 to 1.25. This relatively wide spread in the value of C is probably accounted for mainly by (1) the assumption that the velocity at each of the taps can be calculated by use of the vortex law; and by (2) neglecting the effect of the centrifugal force of the water as it flows around the case. This wide variation in C makes it impossible to use the casing piezometers as a primary method of water measurement but, as pointed out by the author, the piezometers should be rated by an approved method, such as the Allen or Gibson method, and thereafter may be used for measuring purposes.

NOTE.—The paper by Ireal A. Winter, Assoc. M. Am. Soc., C. E. was published in April, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: August, 1933, by Messrs. C. Maxwell Stanley, R. C. Johnon, and E. A. Dow.

[9] With Buffalo, Niagara & Eastern Power Corp., Buffalo, N. Y.

[9a] Received by the Secretary July 26, 1933.

In designing the piezometer and metering equipment, it is important, of course, to know the relative amount of mercury deflection to expect for the full-gate discharge. Some allowance must be made for the possible error involved in the calculation and, as a practical matter, this may be taken care of by adding two more low-pressure taps, one on either side of the main one, as indicated by the author.

Report of Tellers on Second Ballot for Official Nominees

"October 16, 1933

"To the Secretary,

American Society of Civil Engineers:

"The Tellers appointed to canvass the Second Ballot for Official Nominees report as follows:

"Total number of ballots received	1 925
"Deduct:	
Ballots from members in arrears of dues	140
Ballots not signed	10
Ballots from members who have died since voting	1
"Total withheld from canvass	151
"Ballots canvassed	1 774

"For Vice-President, Zone I:	
John P. Hogan	471
Charles M. Spofford	340
Void	25
Blank	11
Total	847

"For Directors, District 1: (Two to be elected)	
O. H. Ammann	443
C. E. Trout	405
Blank	134
Total votes registered	982
Total ballots received	491

"For Director, District 2:	
Frank A. Barbour	206
Blank	9
Total	215

"For Vice-President, Zone IV:	
H. D. Dewell	574
Void	5
Blank	24
Total	603

"For Director, District 6:	
Thomas J. Wilkerson	117
Charles M. Reppert	43
Total	160

"For Director, District 10:	
D. Hewitt Wood	81
Frederick H. McDonald	83
Total	164

"For Director, District 13:	
*T. E. Stanton, Jr	146
*B. A. Etcheverry	146
Void	6
Blank	2
Total	300

"Respectfully submitted,

"E. R. Needles, *Chairman,*

"J. J. Hennebique,
C. D. Thomas,
Hiram Miller,
V. R. Irvine,
Jay D. Moore,

"A. W. Green, Jr.,
Alfred Brahdy,
H. L. Hurd,
Irving Jacobs,
"Tellers."

* According to Article VII, Section 4, of the Constitution, in the event of a tie vote, the names of both candidates shall appear on the final ballot.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from November 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years of active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

BARDIN, WILLIAM JOSEPH, San Jose, Cal. (Age 24.) Refers to H. M. Chadwick, J. C. L. Fish, H. H. Hall, L. B. Reynolds.

BARRETT, EUGENE VINCENT, White Plains, N. Y. (Age 31.) Testing Engr., Civ. Eng. Testing Laboratory, Columbia Univ., New York City. Refers to A. H. Beyer, T. Heatley, G. P. Hevenor, W. J. Krefeld, J. S. Peck, G. E. Strehan.

BEAVIN, BENJAMIN EVERETT, Annapolis, Md. (Age 31.) Asst. Chf. Engr., Annapolis Metropolitan Sewerage Comm. Refers to C. B. Bryant, R. L. Burwell, F. H. Dryden, H. R. Hall, R. B. Morse, G. J. Requardt, A. Wolman.

CHAPMAN, LAUREN BOCK, Berkeley, Cal. (Age 33.) Refers to H. J. Alexander, A. Brahdy, W. M. Paddock, G. Perrine, H. D. Winsor.

CHAYABONGSE, CHAMRAS, Washington, D. C. (Age 24.) Refers to J. B. Babcock, 3d, T. R. Camp, C. M. Spofford.

CLINGERMAN, JOHN WAKEFIELD, Pittsburgh, Pa. (Age 24.) Refers to F. J. Evans, F. M. McCullough.

DOWNS, LEONARD VAUGHN, Denver, Colo. (Age 25.) Jun. Engr., U. S. Bureau of Reclamation. Refers to D. C. Henny, I. E. Houk, W. C. McNown, F. A. Russell, J. L. Savage.

DYSART, ARTHUR, Seattle, Wash. (Age 47.) Archt. and Engr. Refers to E. E. Adams, O. S. Bowen, H. M. Hadley, F. F. Sinks, W. F. Way.

ELY, RODNEY BUSHNELL, Balboa Heights, Canal Zone. (Age 31.) Jun. Engr., The Panama Canal. Refers to B. J. Fletcher, W. B. Godfrey, R. C. Jones, R. L. Klotz, E. S. Randolph.

FRANKLIN, GEORGE EDWARD, Dallas, Tex. (Age 46.) Supt. of Streets and Bridges. Refers to J. A. Focht, J. D. Fowler, K. F. Hoefle, O. H. Koch, J. E. Pirie, W. J. Powell, E. A. Wood.

FRIEDMAN, SAM STANLEY, Brooklyn, N. Y. (Age 21.) Refers to H. R. Codwise, H. P. Hammond, L. F. Rader, E. J. Squire.

GEYER, GEORGE, New York City. (Age 40.) Refers to G. Lindenthal, J. I. Rooney, F. R. Schmid, Z. H. Sikes, H. T. Welty.

GIBSON, COUNT DILLON, Atlanta, Ga. (Age 45.) Major, Engrs., O. R. C., U. S. Army; Prof. and Head of Dept. of Geology, Georgia School of Technology. Refers to R. P. Black, F. H. Frasuer, B. P. McWhorter, W. M. Robinson, Jr., S. B. Slack, F. C. Snow.

GREENE, ROBERT, Bronx, N. Y. (Age 26.) Eng. Asst., Bureau of Design, Borough Pres. of Bronx. Refers to E. F. Fox, E. G. Hooper, C. T. Schwarze.

GRILL, GEORGE MATTHIAS, New York City. (Age 33.) Asst. Engr. on vault surveys, Dept. of Public Works. Refers to N. A. Alexieff, H. de Anguera, N. S. Fuleihan, J. Goodman, R. J. F. Lucchetti, H. B. Machen, F. B. Nelson.

HALPIN, FRANK CORNELIUS, Port Arthur, Tex. (Age 29.) Draftsman with the Texas Co. Refers to L. B. Ryon, Jr., W. E. White.

HENRY, ARTHUR RICHMOND, Pitman, N. J. (Age 22.) Road Supervisor, Borough of Vineland. Refers to H. N. Lendall, J. A. Williams.

JAROS, STANLEY FRANCIS, Columbus, Ohio. (Age 22.) Refers to C. T. Morris, R. C. Sloane.

KINSELLA, JOHN RICHARD, Newport, Ky. (Age 27.) City Engr. of Southgate, Ind., and member of firm, Thiel & Kinsella. Refers to W. L. Glazier, A. Neuffer, R. R. Pyne, F. L. Reed, M. D. Ross.

KIRCHGESSNER, CARLE JAMES, Lansing, Mich. (Age 29.) Bridge Designer with Michigan State Highway Dept. Refers to G. W. Brush, J. H. Cissel, R. E. Everett, L. M. Gram, C. A. Melick, L. L. Stanley, N. Van Eenam.

KJERULF, HANS FREDRIK, Bethlehem, Pa. (Age 31.) With Design and Estimating Dept., McClintic Marshall Corporation. Refers to A. R. Graves, M. G. Hilpert, W. H. Jameson, I. M. Lyse, E. T. Wilson, W. S. Wolfe.

LEIPNER, WALLACE WILLIAM, Cleveland, Ohio. (Age 35.) Industrial Engr. Refers to H. T. Daniels, C. H. Fowler, C. G. French, H. J. Hassler, H. S. Jacoby, R. E. J. Summers.

LEVINE, ERNEST, Los Angeles, Cal. (Age 26.) Supt. and Field Engr. for Contracting Engrs., Inc. Refers to R. M. Beanfield, H. Friedman, R. R. Martel, A. J. Smith, F. Thomas.

LEWIS, CHARLES ALEXANDER, Jr., Phoenix, Ariz. (Age 32.) Lewis Bros., Engrs. & Contrs. Refers to H. H. Brown, F. R. Goodman, V. H. Housholder, E. V. Miller, J. W. Powers.

McGRAW, WILLIAM HAROLD, Oakland, Cal. (Age 27.) Jun. Bridge Constr. Engr., San Francisco-Oakland Bay Bridge. Refers to C. E. Andrew, J. S. Bates, O. R. Bosso, C. R. Harding, G. W. Rear, S. A. Roake, D. R. Warren.

MAY, DAVID CHAPIN, Banning, Cal. (Age 30.) Asst. Engr., Metropolitan Water Dist. of Southern California. Refers to G. E. Baker, E. A. Bayley, J. B. Bond, R. B. Diemer, B. A. Eddy, H. G. Matthews, W. W. Wyckoff.

MILLARD, LESTER WARD, Lansing, Mich. (Age 35.) Acting Bridge Engr., Michigan State Highway Dept. Refers to J. B. Bebb, J. H. Cissel, R. F. Rey, H. A. Shuptrine, J. H. Wasson.

MONTEALEGRE, FRANCISCO GERMAN, Jr., San Francisco, Cal. (Age 29.) Refers to H. A. Harris, Jr., I. M. Ingerson, T. R. Simpson, H. Smitherum, F. H. Tibbetts, G. Zander.

ORR, GERRY MITCHELL, Little Rock, Ark. (Age 25.) With M. K. Orr Contr. Co. Refers to E. S. Anderson, D. B. Cutler, F. A. Gerig, A. M. Lund, J. R. Rhyne, H. C. Sellers, V. H. Smith, W. W. Zass.

PADILLA, GUSTAVO EUGENIO, Humacao, Puerto Rico. (Age 31.) Res. Engr., Dept. of the Interior, Puerto Rico. Refers to M. Font, R. A. Gonzalez, S. Quinones, W. W. Schlecht, E. Totti y Torres.

PHELAN, PAUL FREDERICK, Pittsburgh, Pa. (Age 20.) Refers to A. Diefendorf, L. C. McCandliss.

PRISADSKY, VLADISLAV ALEXANDER, Seattle, Wash. (Age 22.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

TATLOW, RICHARD HENRY, III, Washington, D. C. (Age 27.) Asst. to Administrative Asst. to Administrator of Federal Emergency Administration of Public Works. Refers to T. W. Allen, J. F. Coleman, F. M. Cortelyou, F. R. Dungan, C. L. Eckel, H. J. Gilkey, J. H. Gregory, J. L. Harrington, F. E. Lamphere, C. D. Marx, W. E. Reynolds, A. L. Sherman, C. Shoemaker, W. E. Wilbur.

TEN EYCK, PETER GANSEVOORT, Albany, N. Y. (Age 60.) Chairman, Albany Port Dist. Comm. Refers to G. C. Diehl, L. C. Fritch, E. P. Goodrich, T. Horton, L. F. Loree, P. C. Ricketts, P. Schultze, E. A. Touceda, W. J. Wilgus.

VAN WINGEN, NICO, Long Beach, Cal. (Age 22.) Refers to W. W. Michael, F. Thomas.

WEIRICK, FRITZ HENRY, Olathe, Kans. (Age 28.) Refers to L. E. Conrad, R. D. Goodrich, H. T. Person.

WEST, CARL MILAND, Seattle, Wash. (Age 25.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, O. A. Piper, R. G. Tyler.

WHITE, EDWARD EMELIN, New York City. (Age 24.) Draftsman with Spencer, White & Prentis. Refers to A. Haertlein, H. T. Immerman, G. Paaswell, E. A. Prentis, R. Ridgway, C. B. Spencer.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

CARPENTER, LEWIS VAN, Assoc. M., Morgantown, W. Va. (Elected Nov. 15, 1926.) (Age 37.) Head of Dept. of Civ. Eng., West Virginia Univ.; also Cons. Engr. Refers to H. E. Babbitt, T. J. Blair, Jr., G. P. Boomsliter, W. E. Brooks, R. P. Davis, F. C. Foote, M. G. Mansfield.

CROSBY, HEWITT, Assoc. M., Port Jefferson, N. Y. (Elected Oct. 5, 1909.) (Age 52.) Gen. Mgr. and Chf. Engr., Long Island Shipyard, Inc.; also Cons. Engr. Refers to J. W. Barker, De W. D. Barlow, H. J. Deutschbein, J. W. Doty, G. L. Freeman, A. B. Hager, G. R. Johnson, A. Weymouth.

DUNCAN, DANIEL TOMPKINS, Assoc. M., Greenwood, S. C. (Elected June 9, 1930.) (Age 37.) In private engineering practice. Refers to J. A. Anderson, S. C. Dreyfus, A. E. Johnson, T. K. Legare, F. H. Murray, W. E. Rowe, E. W. Schoder, F. J. Seery, F. R. I. Sweeny.

HAMMELL, CHARLES EDWARD, Assoc. M., Cincinnati, Ohio. (Elected Dec. 14, 1925.) (Age 56.) Senior Engr., U. S. Engr. Office, Office of Sector Engr., Ohio River Sector. Refers to F. B. Duis, C. L. Hall, L. S. Johnson, H. H. Kranz, W. H. McAlpine, L. H. Prell, W. S. Winn.

HOWE, JEROME WILLARD, Assoc. M., Worcester, Mass. (Elected Dec. 15, 1924.) (Age 47.) Prof. and Head of Dept., of Civ. Eng., Worcester Polytechnic Inst. Refers to C. M. Allen, J. W. Childs, D. C. Coyle, A. D. Flinn, A. W. French, W. Hedlund, A. B. Holmstrom, J. A. Johnston, A. J. Knight, R. G. Lingley, C. F. Meyer, E. F. Miner, J. A. Tosi, L. W. West.

KREFELD, WILLIAM JOHN, Assoc. M., New York City. (Elected Junior March 2, 1915; Assoc. M. Jan. 17, 1921.) (Age 41.) Asst. Prof. of Civ. Eng., Columbia Univ. Refers to A. H. Beyer, J. K. Finch, A. G. Hayden, R. P. Miller, G. E. Strehan, W. H. Yates.

LYSE, INGE MARTIN, Assoc. M., Bethlehem, Pa. (Elected April 23, 1928.) (Age 35.) Research Associate Prof. of Eng. Materials, Lehigh Univ. Refers to R. L. Bertin, R. C. Booth, H. F. Gonnerman, F. R. McMillan, F. E. Richart, C. H. Sutherland, H. M. Westergaard.

SKYTTE, JOHANNES, Assoc. M., San Francisco, Cal. (Elected July 12, 1926.) (Age 40.) Hydr. Eng. Designer, Hetch Hetchy Dept., City of San Francisco. Refers to A. L. Alin, F. J. Amweg, L. R. Jorgensen, L. H. Nishkian, F. C. Scobey, L. W. Stocker, D. R. Warren.

SMITH, SCHUYLER MORTON, Assoc. M., Clayton, Mo. (Elected May 31, 1916.) (Age 45.) Bridge Engr., St. Louis County Highway Dept. Refers to B. L. Brown, F. W. Green, F. G. Jonah, A. F. Reichmann, R. A. Van Ness, A. R. Wilson.

THEIMER, OTTO FREDERICK, Assoc. M., Brno, Czechoslovakia. (Elected Jan. 18, 1926.) (Age 40.) Cons. Engr. Refers to A. P. Andersen, W. A. Cuenot, K. Imhoff, G. Lindenthal, F. L. Stuart.

FROM THE GRADE OF JUNIOR

ANDERSON, ANDREW MATTHEW, Jun., White Plains, N. Y. (Elected Oct. 10, 1927.) (Age 30.) Asst. Engr. (structural design), Westchester County Park Comm. Refers to J. Barnett, W. W. Chadsey, E. B. Fitzpatrick, A. G. Hayden, R. M. Hodges, L. G. Holleran, B. L. Weiner.

BELL, EDWARD ARTHUR, Jun., Pitman, N. J. (Elected Oct. 1, 1928.) (Age 27.) Res. Engr. with Clyde Potts, Cons. Engr., New York City. Refers to C. T. Fisher, W. Gavett, M. den H. Kolyn, H. N. Lendall, C. Potts, E. H. Rockwell, S. N. Williams.

CASPAR, FERDINAND EDWARD, Jun., Orange, N. J. (Elected Oct. 24, 1932.) (Age 32.) Asst. Engr., City of Orange. Refers to H. N. Cummings, T. G. Griggs, H. W. Heilmann, W. S. LaLonde, Jr., F. Williams.

HOLDEFER, JOSEPH LUTZ, Jun., Sioux City, Iowa. (Elected Oct. 10, 1927.) (Age 30.) Inspector with Iowa State Highway Comm. Refers to R. A. Caughey, P. D. Cook, J. S. Dodds, E. W. Dunn, A. H. Fuller, M. B. Morris, A. E. Stoddard.

HOOPER, OLCOTT LORIN, Jun., Reading, Pa. (Elected Dec. 15, 1924.) (Age 31.) Asst. Hydr. Engr., W. S. Barstow & Co., Inc. Refers to H. K. Barrows, A. P. Campbell, A. S. Crane, M. Duncan, F. Kurtz, A. F. Smith, Jr.

KAY, CARNIG, Jun., Blandford, Mass. (Elected Oct. 14, 1929.) (Age 31.) Asst. Engr., Springfield Water Works, Cobble Mountain Reservoir. Refers to H. P. Burden, R. D. Chase, C. M. Everett, H. H. Hatch, E. E. Lochridge, A. V. Lynn, J. L. Tighe, E. H. Wright.

NOBLE, JOHN HENRY NEIDLINGER, Jun., Morrisville, Pa. (Elected March 16, 1925.) (Age 31.) Engr., Wire Rope Div., John A. Roebling's Sons Co., Trenton, N. J. Refers to M. Hirschthal, R. S. Johnston, C. M. Segraves, E. R. Taylor, J. L. Vogel, T. S. Williams.

OTTER, JOHN VERNON, Jun., Boise, Idaho. (Elected Feb. 10, 1930.) (Age 28.) Engr. Examiner, Federal Emergency Administration of Public Works. Refers to A. C. Blomgren, I. N. Carter, I. C. Crawford, L. Lee, A. P. Ludberg, C. E. Sherman.

PORTER, EARL LEONARD FRANKLIN, Jun., Alma, Wis. (Elected Oct. 26, 1931.) (Age 32.) Jun. Engr., U. S. Engrs. Refers to F. Bass, E. J. Christenson, A. S. Cutler, O. M. Leland, H. S. Schick, R. C. Vogt.

TUDOR, RALPH ARNOLD, Jun., Oakland, Cal. (Elected July 14, 1930.) (Age 31.) Senior Designing Engr. of Bridges, Div. of Highways, California Dept. of Public Works. Refers to C. E. Andrew, R. E. Davis, F. W. Panhorst, C. H. Purcell, N. C. Raab, G. B. Woodruff.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.